BEARING CAPACITY OF CIRCULAR FOOTING LOCATED ON RANDOMLY REINFORCED SAND BED

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CERTIFICATE

This is to certify that the thesis entitled, "**BEARING CAPACITY OF CIRCULAR FOOTING LOCATED ON RANDOMLY REINFORCED SAND BED**" submitted by **SAILENDRA KUMAR HALWAI** in partial fulfillments for the requirements for the degree of Bachelor of Technology 20010-11 in Civil Engineering at **National Institute of Technology**, **Rourkela** is an authentic work carried out by him under my supervision and guidance.

To the best of my knowledge, the matter embodied in this report has not been submitted to any other University / Institute for the award of any Certificate.

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ABSTRACT

Earth reinforcement is an effective and reliable technique for increasing the strength and stability of soils. The technique is used today in a variety of applications ranging from retaining structures and embankments to sub grade stabilization beneath footings and pavements. Reinforcement can vary greatly; either in form (strips, sheets, grids, bars, or fibers), texture (rough or smooth), and relative stiffness (high such as steel or low such as polymeric fabrics). In past practice reinforcements have typically consisted of long, flexible, galvanized steel strips with either a smooth or ribbed surface. Most field research to date on the mechanics of reinforced earth has tended to focus on high modulus, steel strips. **(Wasti Y Butun MD [1997])**However, randomly distributed fiber reinforced soils have recently attracted increasing attention in geotechnical engineering.

In comparison with systematically reinforced soils, randomly distributed fiber reinforced soils exhibit some advantages. Preparation of randomly distributed fiber reinforced soils represents soil stabilization by admixture. Discrete fibers are simply added and mixed with the soil, much like cement, lime, or other additives. Randomly distributed fibers offer strength isotropy and limit potential planes of weakness that can develop parallel to oriented reinforcement.**(Yetimoglu T Salbas O [2003])**

In the current study bearing capacities for sand specimens containing fiber contents 0%, 0.1%, 0.25%, 0.5%, 0.75% and having relative densities 40%, 55%, 73%, 88% were prepared and tested.

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Chapter 1

INTRODUCTION

1.1 BEARING CAPACITY OF FOOTING ON SEMI-INFINITE SOIL

The lowest part of a structure is generally referred to as the foundation. Its function is to transfer the load of the structure to the soil on which it is resting. A properly designed foundation transfers the load throughout the soil without overstressing the soil. Overstressing the soil can result in either excessive settlement or shear failure of the soil, both of which cause damage to the structure. Thus, geotechnical and structural engineers who design foundations must evaluate the bearing capacity of soils.

1.2 MODES OF FOUNDATION FAILURE

Foundation is that part of the structure which is in direct contact with the soil and transmits load directly to the soil. Prior to the application of load, the soil beneath the base of the footing is in elastic equilibrium. As the load is applied settlement occur which is proportional to the load. With the increase in loading, settlement progressively increases, and the soil transforms from the state of elastic equilibrium to plastic equilibrium and thus the distribution of soil reaction changes and failure of soil occurs. There are three principal modes of shear failure i.e. general shear failure, local shear failure and punching shear failure depending upon the relative compressibility and characteristics of soil. General shear failure occurs in relatively incompressible soil with finite shearing strength. The failure is accompanied by considerable bulging of on the soil surface. Local shear failure occurs in relatively compressible soil. The failure is accompanied by visible sheared zone after bulging has taken place. Punching shear failure takes place due to the relatively great compressibility of soil and may be evaluated by determining the rigidity index of the soil.

1.3 BEARING CAPACITY THEORIES

A number of equations based on theoretical analysis and experimental investigations are available to determine the ultimate bearing capacity equation.

1.3.1 TERZAGHI'S ANALYSIS[12]:

Main assumption made by Terzaghi was that the soil behaves like an ideally plastic material (This concept was initially developed by Prandtl). Terzaghi analysed the failure of a shallow continuous footing (L/B $=\infty$) and then suggested mo difications for isolated square, rectangular and circular footings. The three cases considered by him are (1) smooth base of a footing resist on an ideal soil surface, (2) Rough base of a footing resting on an ideal soil surface and (3) Rough base of a footing resting at a level below the ground surface.

Terzaghi has neglected the shearing resistance of the soil above the base of the footing. The soil above the base of the footing is substituted by an equivalent surcharge **($q=\gamma*D_f$),** where **γ** = unit weight of soil above the base of the footing. According to Terzaghi, the soil mass above the failure surface consists of three zones:

Zone I: Because of friction and adhesion between the soil and the base of the footing, this zone cannot spread laterally. It moves downward as an elastic wedge and the soil in this zone behaves as if it is a part of the footing. The two sides of the wedge ac and bc make angle Φ with the horizontal[2].

Zone II: The zones aef and bed are under this zone, which are called zones of radial shear. The soil in this zone is pushed into zone III[2].

Zone III : These are the two passive Rankine zones, boundaries of which make angles $(45° - \varphi/2)$ with the horizontal[2].

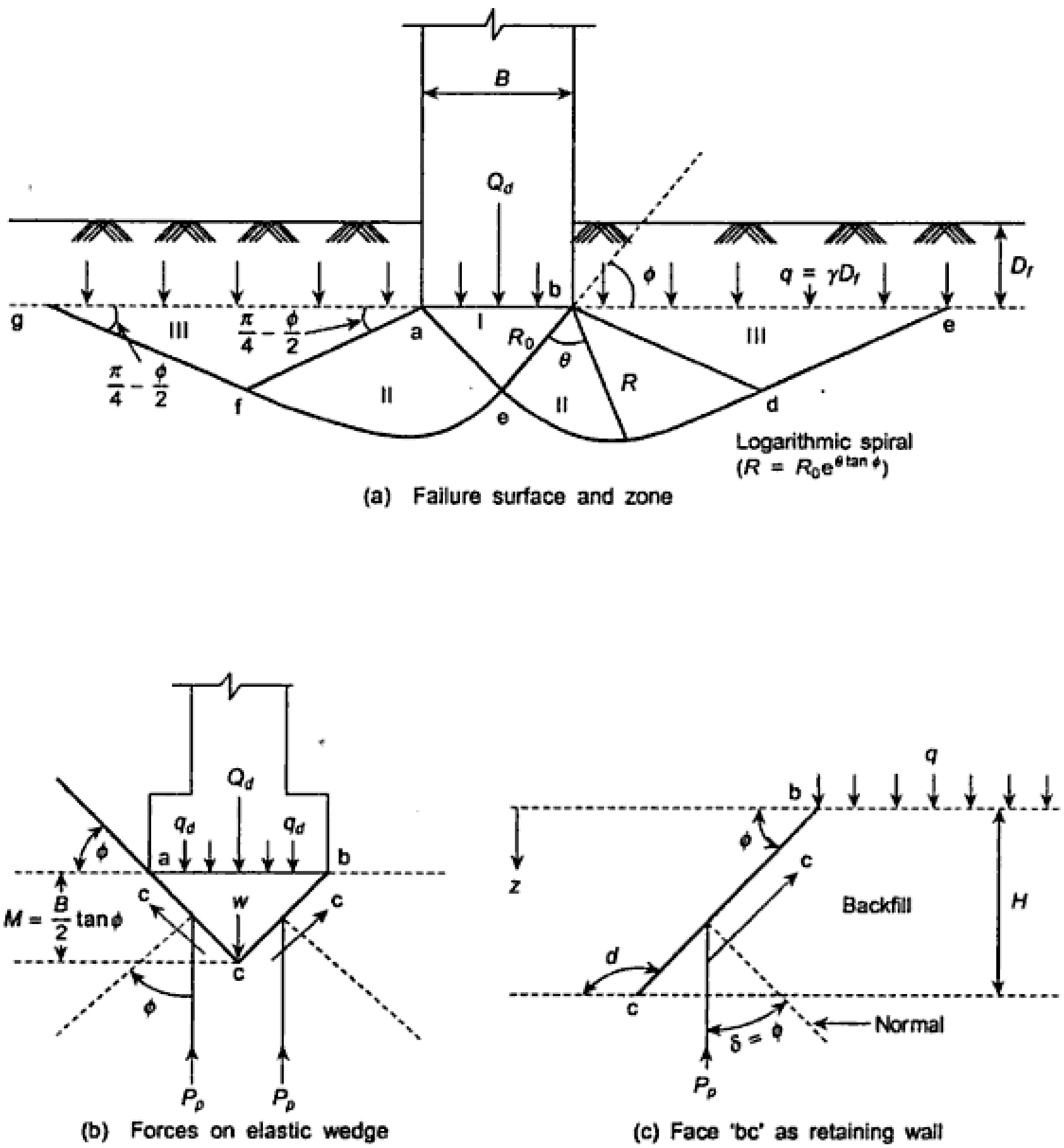

Figure 3.1 Terzaghi's Bearing Capacity Analysis[14]

The equation for ultimate bearing capacity can be expressed as,

$$q_f = cN_c + \bar{\sigma} N_q + 0.5\gamma B N_\gamma$$

Where, γ corresponds to surcharge unit weight and unit weight of the soil under footing.

This is known as general bearing capacity equation N_γ, N_q andN_c are Terzaghi's dimensionless bearing capacity factors due to the soil weight, surcharge and cohesion respectively. There have numerical values depend upon the value of the angle of internal friction of the soil.

$$N_q = \frac{a^2}{2\cos^2(45° + \varphi/2)}$$

$$N_c = \left(\frac{a^2}{2\cos^2(45° + \varphi/2)} - 1\right)\cot\varphi$$

$$N_\gamma = \frac{\tan\varphi}{2}\left(\frac{K_{p\gamma}}{\cos^2\varphi} - 1\right)$$

Where, $K_{p\gamma}$ = passive earth pressure coefficient, dependent on φ,

$$a = e^{(0.75\pi - \varphi/2)\tan\varphi}$$

and φ = Angle of internal friction

Terzaghi has determined these bearing capacity factors as functions of the angle of internal friction φ as given in the table 1.1 in the next page.

Table 1.1TERZAGHI'S BEARING CAPACITY FACTORS [4]

$\varphi°$	N_c	N_q	N_γ	$\varphi°$	N_c	N_q	N_γ
0	5.7	1	0	26	27.09	14.21	9.84
1	6	1.1	0.01	27	29.24	15.9	11.6
2	6.3	1.22	0.04	28	31.61	17.81	13.7
3	6.62	1.35	0.06	29	34.24	19.98	16.18
4	6.97	1.49	0.1	30	37.16	22.46	19.13
5	7.34	1.64	0.14	31	40.41	25.28	22.65
6	7.73	1.81	0.2	32	44.04	28.52	26.87
7	8.15	2	0.27	33	48.09	32.23	31.94
8	8.6	2.21	0.35	34	52.64	36.5	38.04
9	9.09	2.44	0.44	35	57.75	41.44	45.41
10	9.61	2.69	0.56	36	63.53	47.16	54.36
11	10.16	2.98	0.69	37	70.01	53.8	65.27
12	10.76	3.29	0.85	38	77.5	61.55	78.61
13	11.41	3.63	1.04	39	85.97	70.61	95.03
14	12.11	4.02	1.26	40	95.66	81.27	115.31
15	12.86	4.45	1.52	41	106.81	93.85	140.51
16	13.68	4.92	1.82	42	119.67	108.75	171.99
17	14.6	5.45	2.18	43	134.58	126.5	211.56
18	15.12	6.04	2.59	44	151.95	147.74	261.6
19	16.56	6.7	3.07	45	172.28	173.28	325.34
20	17.69	7.44	3.64	46	196.22	204.19	407.11
21	18.92	8.26	4.31	47	224.55	241.8	512.84
22	20.27	9.19	5.09	48	258.28	287.85	650.67
23	21.75	10.23	6	49	298.71	344.63	831.99
24	23.36	11.4	7.08	50	347.5	415.14	1072.8
25	25.13	12.72	8.34				

When, $\varphi = 0$, in case of pure cohesive soils,

$N_c = \frac{3}{2}\pi + 1 = 5.71$ (Calculated by applying L'Hospital's Rule to the N_c function, because

With $\varphi = 0$ the $N_c = 0$.)

$N_q = 1$,

$N_\gamma = 0.$

In order to take into account the shape of the footing (i.e. strip, round, square, etc.), Terzaghi used only

The equation for ultimate bearing capacity for circular footing can be expressed as,

$$q_f = 1.3cN_c + \bar{\sigma}\,N_q + 0.3\gamma BN_\gamma$$

The equation for ultimate bearing capacity for square footing can be expressed as,

$$q_f = 1.3cN_c + \bar{\sigma}\,N_q + 0.4\gamma BN_\gamma$$

Terzaghi's method of analysis of the bearing capacity of a cohesive soil is independent of the width of the footing. The settlement, however, of a cohesive soil is inversely proportional to the width 'b' of the footing.

The allowable bearing capacity, $q_{allowable}$ of a cohesive soil is obtained by dividing the ultimate bearing capacity, $q_{u,}$ by a factor of safety, say f.

Terzaghi has further defined two types of failures. Before loading the soil is in a state of elastic equilibrium. When a load greater than critical load is applied, the soil gradually passes to a state of plastic equilibrium. For this transition from elastic to plastic state there may be either loacal shear failure or general shear failure.

1.3.2 MEYERHOF'S THEORY:

Meyerhof extended Terzaghi's analysis of the plastic equilibrium of the surface footing to shallow and deep foundations, considering the shear strength of overburden. Figure 1.2, shows the failure mechanism for shallow and deep foundations according to both Terzaghi and

Meyerhof's analysis. In the Meyerhof's analysis, *abd* is the elastic zone, *bde* is the radial shear zone and *befg* is the zone of mixed shear in which shear varies between radial and plane shear, which depend upon the depth and roughness of the foundation. The plastic equilibrium in all these zones is established from the boundary conditions starting from the foundation shaft. To make analysis simpler, Meyerhof introduced a parameter β, the angle to define the line *bf*, joining point *b* to *f* where the boundary failure slip line intersects the soil surface. The resultant effects of the wedge *bfg* are represented by normal stress and tangential stress, p_0 and s_0 on *bf*. The plane *bf* is termed as the *equivalent free surface,* and p_0

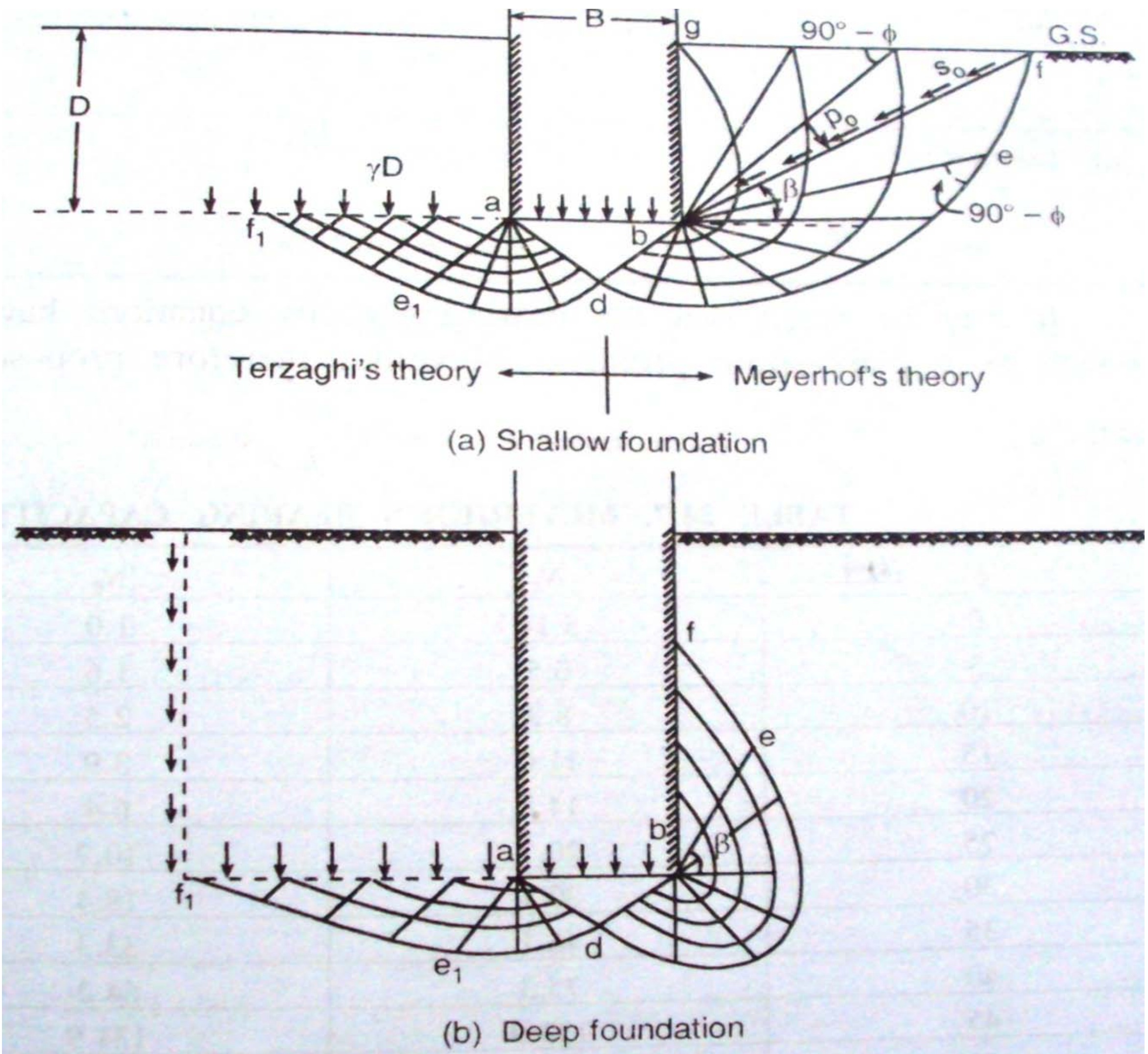

Figure 1.4 Meyerhof's Analysis[12]

and s_0 are termed as the *equivalent free surface stresses.* The value of β increases with depth, and becomes 90° for deep foundations.

The equation for ultimate bearing capacity (taking into account the shape, depth and inclination factors) can be expressed as,

Vertical Load: $q_f = cN_c s_c d_c + \bar{\sigma} N_q s_q d_q + 0.5\gamma B N_\gamma s_\gamma d_\gamma$

Inclined Load: $q_f = cN_c d_c i_c + \bar{\sigma} N_q d_q i_q + 0.5\gamma B N_\gamma S_\gamma d_\gamma$

Above expression is of same form as that of Terzaghi, butN_c, N_q and N_γ now depend upon the depth and shape of the foundation and the angle of internal friction and the roughness of the base.

$$N_q = e^{\pi \tan \varphi} \tan^2(45° + \varphi/2)$$

$$N_c = (e^{\pi \tan \varphi} \tan^2(45° + \varphi/2) - 1) \cot \varphi$$

$$N_\gamma = (e^{\pi \tan \varphi} \tan^2(45° + \varphi/2) - 1) \tan 1.4\varphi$$

Table 2.2 MEYERHOF'S BEARING CAPACITY FACTORS[12]

$\varphi°$	N_c	N_q	N_γ
0	5.14	1.0	0
5	6.5	1.6	0.1
10	8.3	2.5	0.4
15	11.0	3.9	1.1
20	14.8	6.4	2.9
25	20.7	10.7	6.8
30	30.1	18.4	15.7
35	46.1	33.3	37.1
40	75.3	64.2	93.7
45	133.9	134.9	262.7
50	266.9	319.0	873.7

1.3.3 HANSEN'S MODIFICATION[12]:

Brinch Hansen (1960) modified the equation of Terzaghi by including five new variables, namely, the (i) shape factor 's', (ii) depth factor 'd', (iii) inclination factor 'i', (iv) ground factor 'g' and (v) base factor 'b'

and can be expressed as,

$$q_u = cN_c s_c d_c i_c g_c b_c + \sigma_0 N_q s_q d_q i_q g_q b_q + 0.5\gamma B N_\gamma s_\gamma d_\gamma i_\gamma g_\gamma b_\gamma$$

Where,

σ_0 = effective overburden pressure at foundation level, s = shape factor, to consider the effect of the shape of the foundation in developing a failure surface,

d = depth factor to account for the embedment depth and the extra shearing resistance in the top soil,

i = inclination factor to account for both horizontal and vertical components of foundation loads,

g = ground factor,

b = base factor,
γ = density of soil below the foundation level.

Hansen's recommendation for the bearing capacity factors are,

$$N_q = e^{\pi \tan \varphi} \tan^2(45° + \varphi/2)$$

$$N_c = (e^{\pi \tan \varphi} \tan^2(45° + \varphi/2) - 1) \cot \varphi$$

$$N_\gamma = 1.8\ (e^{\pi \tan \varphi} \tan^2(45° + \varphi/2) - 1) \tan \varphi$$

Equations for depth and shape factors are,

$$d_c = \frac{1 + 0.35}{\frac{B}{D_f} + \frac{0.6}{1+7\tan^4 \varphi}}$$

$$\mathrm{d_q} = \mathrm{d_c} - \frac{(\mathrm{d_c} - 1)}{\mathrm{N_q}}$$

$$s_c = 1 + (0.2 + \tan^6 \varphi)\ {}^{B}/_{L}$$

Where L = Length of the rectangular foundation

$$s_q = s_0 - (s_0 - 1)/N_q$$

$$s = 1 - {}^{1}\!/_{2}\,(0.2 + \tan^2 \varphi)\, {}^{B}\!/_{L}$$

1.4 PRINCIPLES OF REINFORCED EARTH

Soil mass is generally a discrete system which consists of soil grains. It cannot bear tensile stresses and this is particularly true in the case of cohesion less soil like sand. Such soils cannot be stable on steep slopes and relatively large strains are caused when external loads are imposed on them. Reinforced earth is a composite material, a combination of soil and reinforcement suitably placed to bear the tensile stresses developed and also to improve the resistance of soil in the direction of greatest stress. The presence of reinforcement alters the stress field giving a restraint mostly in the form of friction or adhesion so thatless strainis induced and tension is avoided. Inclusions like discrete &short fibers placed random or in definite layers will also impart additional resistance by way of cohesion and friction.

1.5 EFFECT OF REINFORCEMENT ON SOIL

1.5.1 TRANSFER OF FORCE FROM SOIL TO REINFORCEMENT

Figure 1.3 shows cohesion less soil mass reinforced with a flat strip. The forces at the ends of the strip are not equal when there is transference of force by friction to the soil mass (vidal, 1969). If the average vertical stress in the soil is $\sigma_v$in the region, the difference between the forces at the end of a reinforcing element AB of length ‘dl’can be expressed by

$$dP = \sigma_v . 2w . dl . \tan \varphi_\mu$$

Where w is the width of the reinforcement and φ_μ is the angle of friction between the reinforcement and the soil.

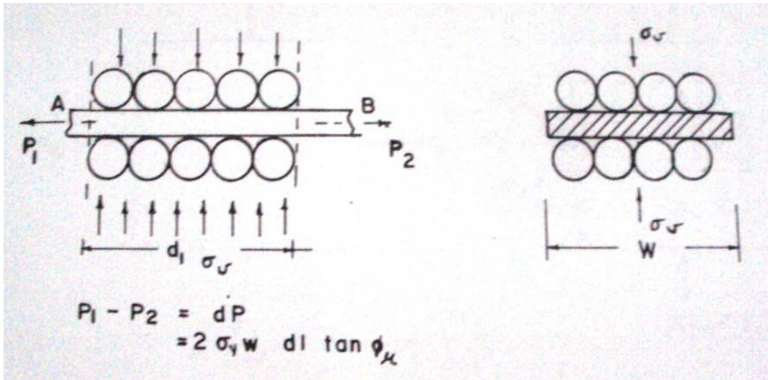

Figure 1.3 Stress Transfer By Soil Reinforcement[13]

Therefore, if we consider a soil system with reinforcement at a spacing of $'\Delta h'$and $'\Delta v'$ as shown in the figure 1.4 the effect of the reinforcement on the soil mass will be to restraint by imposing an additional stress of

$$\Delta\sigma_3 = \Delta h\left(\frac{dp}{\Delta v}\right)$$

This restraint on the soil system increases the resistance of the soil to failure under applied stress.

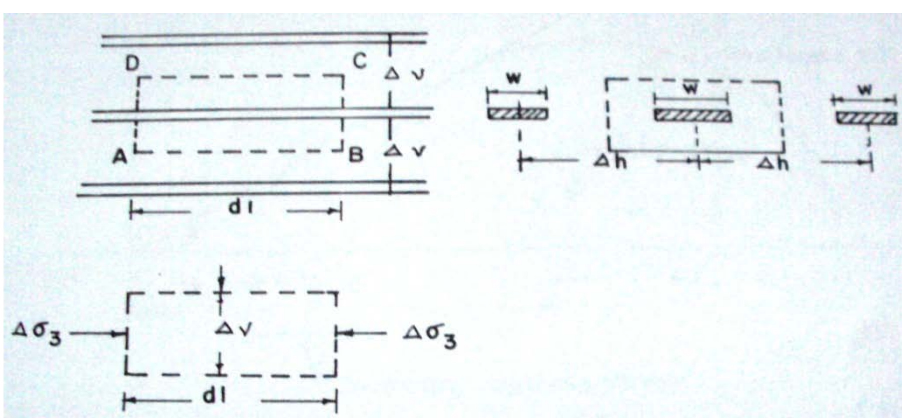

Figure 1.4Confining Stress on Soil by Reinforcement[13]

1.5.2 EQUIVALENT CONFINING STRESS CONCEPT

Fig. 1.5 shows the comparison of failure stresses on two soils, (i) unreinforced and (ii) reinforced. The increase in the deviatoric stress is seen to be $\Delta\sigma_3$times k_p, where k_p is the coefficient of passive earth pressure equal to $\tan^2(45°+\varphi/2)$ and $\Delta\sigma_3$is the equivalent confining stress on the sand imposed by the reinforcement (Yang, 1972).

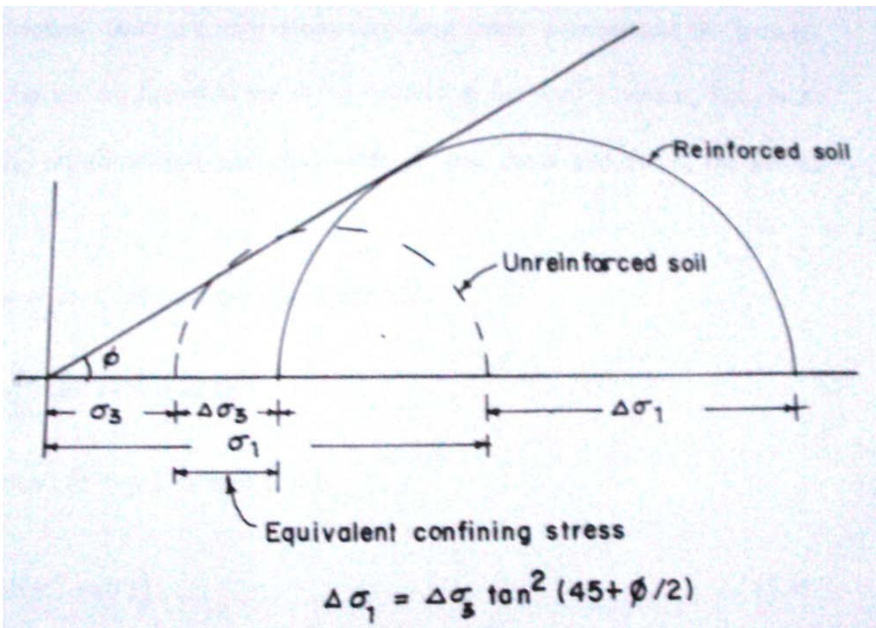

Figure 1.5 Equivalent Confining Stress Concept[13]

1.5.3 PSEUDO COHESION CONCEPT[12]

This concept (Schlosser and Long, 1974) proposes that the reinforcement induces an anisotropic or pseudo cohesion to the soil which depends on the spacing and strength of the reinforcement as shown in the fig. 1.6. The increase in deviator stress at failure is

$$\Delta\sigma_1 = 2c.\tan(45° + \frac{\varphi}{2})$$

Where ‘c’ is the pseudo cohesion induced in the soil and φ is the angle of friction. Both the equivalent confining stress concept and the pseudo cohesion concept are linked to the stress induced in the reinforcement. If α_f is the force in the reinforcement per unit width of soil mass and Δv is the vertical spacing.

$\frac{\alpha_f}{\Delta v}$ isthe equivalent confining pressure $\Delta\sigma_3$

And $\Delta\sigma_1 = \frac{\alpha_f}{\Delta v}.\,tan^2(45° + \frac{\varphi}{2})$

Or $\Delta\sigma_1$ = 2c. tan$(45° + \frac{\varphi}{2})$ which yields

$$c = \frac{\alpha_f}{2\Delta v}.\tan(45° + \frac{\varphi}{2})$$

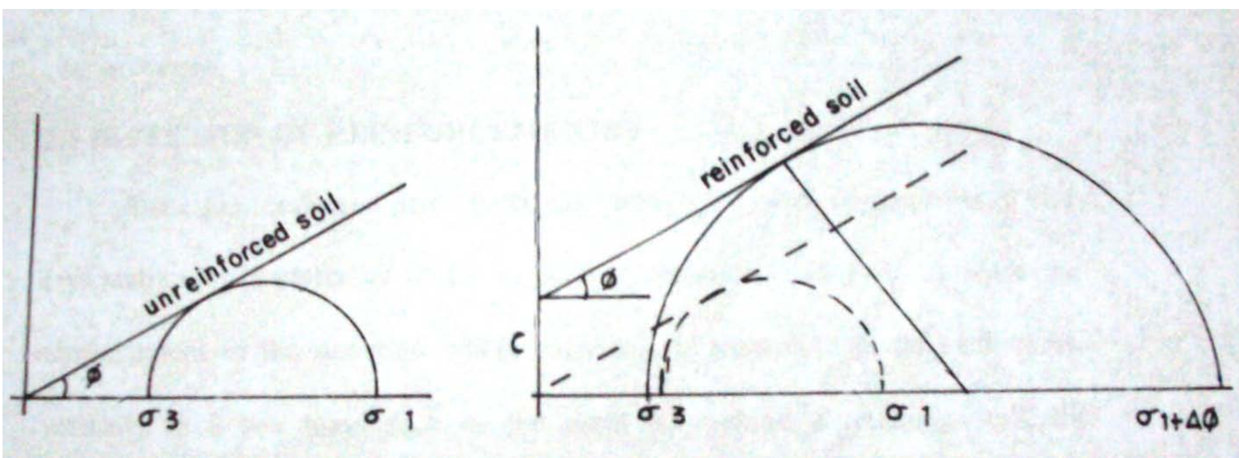

Figure 1.6Pseudo Cohesion Concept[13]

The value of $\alpha_f$is same as the tensile strength of the reinforcement, if the reinforcement fails by breakage or the maximum force transferred by friction between the soil and reinforcement, if there is bond failure and reinforcement pulls off.

In the above mentioned concept, it is necessary that the layer of reinforcement must be close enough so that there is effective transfer of stress by friction or adhesion as the case may be and hence the granular soils of high relative density are particularly suitable for use in reinforced earth.

The concept outlined above can also hold good for cohesive soils to a very limited extend only since the adhesion of the clay to reinforcement is small and its effect on restraint does not have a multiplying effect as in granular materials.

1.6 FIBER REINFORCED SOIL

Randomly distributed fibers reinforced soil (RDFS) is among the latest ground improvement techniques in which fibers of desired type and quantity are added in the soil, mixed randomly and laid in position after compaction, a method similar to conventional stabilization techniques. RDFS is different from other soil-reinforcing methods in its orientation. Unlike reinforced earth, in RDFS fibers are mixed randomly in soil thus making a homogenous mass and maintain the isotropy of the soil mass.

1.7 ADVANTAGES OF FIBER-REINFORCED SOIL[14]

Randomly distributed fiber-reinforced soil have many advantages to be considered:

- Increase in shear strength with maintenance of strength isotropy.
- Beneficial for all type of soils (i.e. sand, silt and clay).
- Reduced Post peak strength loss.
- Increased ductility.
- Increased seismic performance.

- No catastrophic failure
- Great capacity to use natural or waste material such as coir fibers and recycled waste plastic strips.
- Provide erosion control and facilitate vegetation development
- Reduce shrinkage and swell pressures of expansive soil.
- No appreciable change in permeability.
- Unlike lime, cement, and other chemical stabilization methods, the construction using fiber-reinforcement is not greatly affected by weather conditions.
- Fiber-reinforcement has been reported to be helpful in discarding the shallow failure on the slope face and thus reducing the cost of maintenance.

1.8 MECHANISM OF REINFORCEMENT

Randomly oriented discrete inclusions added into soil improve its load-deformation behavior by interacting with the soil particles mechanically through surface friction and also by interlocking. The function of the bond is to transfer the stress from the soil to the discrete inclusions by mobilizing the tensile strength of discrete fibers. Thus, fiber-reinforcement works as frictional and tension resistant element.

1.9 TYPES OF FIBERS

Fibers can be classified in two main categories: Synthetic fiber and Natural Fiber

(i) Synthetic fibers:The various types of synthetic fibers are nylon, plastic, polypropylene, glass, asbestos, etc. These are generally preferred than the natural fibers because of their higher strength and resistance.

(ii) The various types of natural fiber available in India are coir, sisal, jute, hemp, bhabar, munja, bamboo, banana. In order to minimize the cost of the reinforced soil, locally available fibers are considered in design. Due to its low strength and lack of durability, natural fibers are not used widely for reinforcements but are preferred for erosion control as they are eco-friendly.

1.10 DIRECTION OF REINFORCEMENTS

As the reinforced earth functions through a system of restraints of soil deformation by transfer of stress to reinforcement, it is logical to place the reinforcement in the direction where stress is maximum in an earth mass. Normally in a soil systemthe reinforcement is most effective in the horizontal direction.

However, by introducing reinforcement and increasing strain, shear stresses are induced in a horizontal plane and consequently horizontal and vertical planes cease to be the principal planes and direction of maximum strains.

Fibers can be oriented or mixed randomly in the soil. In oriented case, the fibers are placed within the soil at specific positions and directions. In case of random orientation, fibers are mixd with the soil and placed within the probable shear zone.

1.11 FACTORS AFFECTING THE STRENGTH AND PROPERTIES OF RANDOMLY DISTRIBUTED FIBER-REINFORCED SOIL

- Type of soil
- Type of Fiber: monofilament or fibrillated
- FIBER CONTENT: EXPRESSED IN % WITH RESPECT TO WEIGHT OF SOIL.

- DENIER OF FIBER
- FIBER LENGTH
- ASPECT RATIO
- FIBER-SOIL SURFACE FRICTION

1.12 MODES OF FAILURES IN REINFORCED EARTH STRUCTURES[14]

The following modes of failure have been observed in reinforced earth structures.

(I) Shear failure of the soil above the upper most layer of reinforcement: this mode of failure is possible if the depth to the topmost layer of reinforcement is sufficiently large so as to form an effective boundary into which the shear zones cannot penetrate.

(II) Reinforcement pullout failures: the type of failure occurs for reinforcements placed at shallow depths below the footing and/or reinforcements which have insufficient anchorage

(III) Reinforcement tension failure: this type of failure occurs in the case of long and shallow reinforcements for which the functional pullout resistance is more than the tensile strength.

Chapter 2

LITERATURE REVIEW

LITERATURE REVIEW:

In comparison to systematically reinforced soils, less information has been reportedon randomly distributed fiber-reinforced soils in the literature. However, anincreasing number of experimental and numerical studies on the subject have beenconducted by several researchers in the past few decades (e.g., Hoare, 1979; Grayand Ohashi, 1983; Freitag, 1986; Gray and Al-Refeai, 1986; Maher and Gray, 1990;Ranjan et al., 1996; Bauer and Oancea, 1996; Michalowski and Zhao, 1996; Wastiand Butun, 1996; Consoli et al., 1998; Kumar et al., 1999; Santoni et al., 2001;Kaniraj and Havanagi, 2001).

These previous studies indicate that stress–strain–strength properties of randomly distributed fiber-reinforced soils are also a functionof fiber content, aspect ratio, and fiber-surface friction along with the soil and fiberindex and strength characteristics.

Gray Donald H, OhashiHarukazu.[1983] Direct shear tests were run on a dry sand reinforced with differenttypes of fibers. Both natural and synthetic fibers plus metal wires were tested.Experimental behavior was compared with theoretical predictions based on aforce equilibrium model of a fiber reinforced sand. Test results showed thatfiber reinforcement increased the peak shear strength and limited post peakreductions in shear resistance. The fiber reinforcement model correctly predictedthe influence of various sand-fiber parameters through shear strengthincreases that were: (1) Directly proportional to concentration or area ratio offibers; (2) greatest for initial fiber orientations of 60° with respect to the shearsurface; and (3) approximately the same for a reinforced sand tested in a looseand dense state, respectively. The findings of this study are relevant to suchdiverse problems as the contribution of roof reinforcement to the stability ofsandy, coarse textured soils in granitic slopes, dune and beach stabilization bypioneer plants, tillage in root permeated soils, and soil stabilization with lowmodulus, woven fabrics.

Wasti Y., Butun M.D., [1996]. A series of laboratory model tests on a strip footing supported by sandreinforced by randomly distributed polypropylene fiber and mesh elements wasconducted in order to compare the results with those obtained from unreinforcedsand and with each other. For conducting the model tests, uniformsand was compacted in the test box at its optimum moisture content andmaximum dry density. Three types of reinforcement, two sizes of meshelements having the same opening size and one size of fiber element cut fromthe meshes, were used in varying amounts in the tests. Results indicated thatreinforcement of sand by randomly distributed inclusions caused an increase inthe ultimate bearing capacity values and the settlement at the ultimate load ingeneral. The effectiveness of discrete reinforcing elements was observed todepend on the quantity as well as the shape of the inclusions. The larger meshsize was found to be superior to other inclusions considering the ultimatebearing capacity values. For the mesh elements there appears to be an optimuminclusion ratio, whereas fibers exhibited a linearly increasing trend onthe basis of an increase in ultimate bearing capacity for the range of reinforcement amounts employed.

McGown, Andrews & Hytiris(1985) Drained triaxial test and model footing tests were done. Result showed that mesh increased the deviator stress developed at all strains, even at very small strains, and the peak stresses in the sand-mesh mixture occurred at slightly higher axial strains than for the sand alone. Very large improvements were obtained at all strain levels which were similar to triaxial tests in terms of both strength and deformation characteristics. Recoverable settlement plot shows that where a layer of sand –mesh mixture was present, almost 20% of the imposed vertical settlement was recovered, which was 4 times that for the soil alone.

Yetimoglu T Salbas O [2003]A study was undertaken to investigate the shear strength of sands reinforced with randomlydistributed discrete fibers by carrying out direct shear tests. The effect of the fiberreinforcement content on the shear strength was investigated. The results of the testsindicated that peak shear strength and initial stiffness of the sand were not affectedsignificantly by the fiber reinforcement. The horizontal displacements at failure were alsofound comparable for reinforced and unreinforced sands under the same vertical normalstress. Fiber reinforcements, however, could reduce soil brittleness providing smaller loss ofpost-peak strength. Thus, there appeared to be an increase in residual shear strength angle ofthe sand by adding fiber reinforcements.

HatafN., Rahimi M.M., [2005]. A series of laboratory model tests has been carried out to investigate the using of shredded waste tires as reinforcement to increasethe bearing capacity of soil. Shred content and shreds aspect ratio are the main parameters that affect the bearing capacity.Tire shreds with rectangular shape and widths of 2 and 3 cm with aspect ratios 2, 3, 4 and 5 are mixed with sand. Five shred contentsof 10%, 20%, 30%, 40% and 50% by volume were selected. Addition of tire shreds to sand increases BCR (bearing capacity ratio)from 1.17 to 3.9 with respect to shred content and shreds aspect ratio. The maximum BCR is attained at shred content of 40% anddimensions of 3 · 12 cm. It is shown that increasing of shred content increases the BCR. However, an optimum value for shred contentis observed after that increasing shreds led to decrease in BCR. For a given shred width, shred content and soil density it seems that aspect ratio of 4 gives maximum BCR.

Chapter 3

EXPERIMENTAL INVESTIGATIONS

3.1TEST SAND PREPARATION

Relatively uniformly graded sand was used in this study. The sand used in the test was cleaned and sieved by 200 micron sieve. This sand is classified as SW by Unified Soil Classification System (USCS). The particle size distribution of the sand is shown in Fig. 3.1. It had a mean grain diameter (D_{50}) of 0.55 mm. Various tests are performed to obtain the engineering properties of sand. Those are listed in Table 3.4.

3.2 DETERMINATION OF PROPERTIES OF THE TEST SAND

3.2.1 GRAIN SIZE DISTRIBUTION

Materials and Equipments used:

(i) Balance accurate to 1 g,

(ii) Set of IS sieves: 2 mm, 1 mm , 600 micron, 425 micron, 212 micron

(iii) Thermostatically controlled oven,

(iv) Trays, Sieve Brushes and a wire brush

Table 3.1 Data and Observation Sheet for Sieve Analysis

Sl. No.	IS sieve	Particle size D (mm)	Mass Retained (g)	Cumulative retained (g)	Cumulative % retained	Cumulative % finer
1	2 mm	2	0.4	0.4	0.13	99.87
2	1 mm	1	19.2	19.6	6.53	93.47
3	600 micron	0.6	100.5	120.1	40.03	59.97
4	425 micron	0.425	110.3	230.4	76.8	23.2
5	212 micron	0.212	68	298.4	99.47	0.53

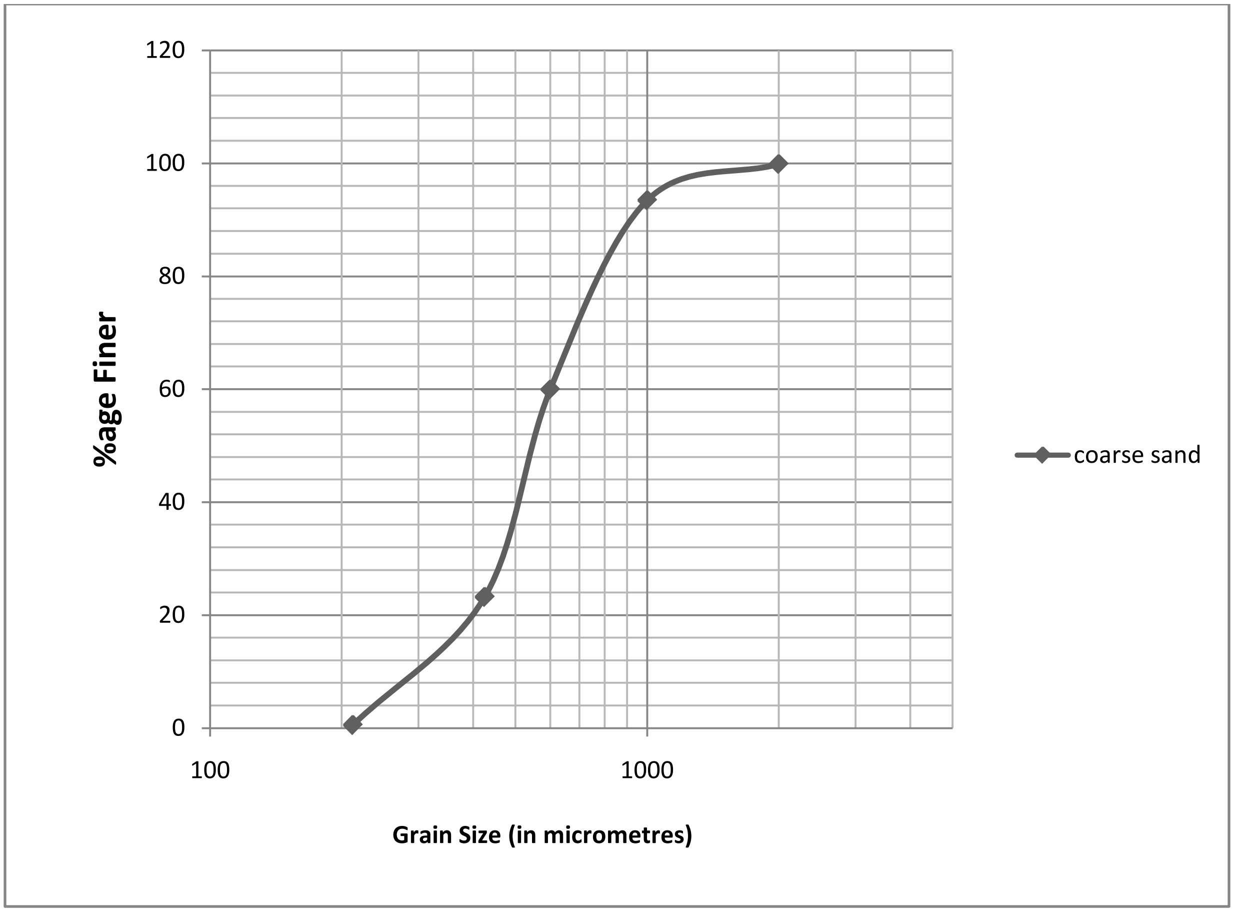

Figure 3.1 Grain Size Distribution

3.2.2 DETERMINATION OF SPECIFIC GRAVITY

Materials and Equipments:

(i) Pycnometer, with a stopper,

(ii) Balance sensitive to 0.01 g,

(iii) Glass rod,

(iv) Distilled water

Table 3.2 Data & Observation Sheet

Test No.	1	2	3
Temperature (deg.)	27	27	27
Bottle No.	8	11	2
Weight of Sp. Gr. Bottle W_1(g)	122.93	124.12	116.53
Weight of Sp. Gr. Bottle + Soil W_2(g)	172.93	174.12	166.53
Weight of Sp. Gr. Bottle + Soil +Water W_3(g)	409.84	411.34	404.04
Weight of Sp. Gr. Bottle + Water W_4(g)	378.89	380.28	372.9
Specific Gravity of Soil (G)	2.62	2.63	2.65
Average	2.63		

3.2.3 DETERMINATION OF MAXIMUM AND MINIMUM DENSITY

Materials and Equipments:

(i) Mould (15 cm dia, 17 cm height)

(ii) Balance sensitive to 1g

(iii) Dynamic Shaker

(iv) Funnel

Table 3.3 Data Sheet for Max. & Min. Density

Specimen for	Wt. of Sand (g)	Volume of the mould (cc)	Density(g/cc)	Void ratio
Maximum Density	4886	3004.15	1.6264	0.6171
Minimum Density	4166	3004.15	1.3867	0.8966

Table 3.4 Properties of Sand

Property	Value
Specific gravity	2.63
Maximum dry unit weight(kN/m^3)	15.95
Minimum dry unit weight(kN/m^3)	13.60
Maximum void ratio	0.897
Minimum void ratio	0.617
Effective grain size D_{10} (mm)	0.30
D_{60} (mm)	0.60
D_{30} (mm)	0.47
Co-efficient of Uniformity (C_U)	2.00
Co-efficient of Curvature (C_C)	1.227

3.3 TEST REINFORCEMENT :

Coir fiber is used as reinforcement during the experimental work. It was cut into small sizes(about 12-15 mm). Fiber properties of coir are given in Table 3.5.

Figure 3.2 Reinforcement used (coir)

Table 3.5 Fiber Properties of Coir[16]

Quantity	Value	Unit
Young's modulus	4000 - 5000	MPa
Tensile strength	140 - 150	MPa
Elongation	15 - 17.3	%
Thermal conductivity	0.047 - 0.047	W/m.K
Density	1.15 - 1.33	kg/m^3
Water absorption	10 - 0	%

3.4 WORK PROCEDURE

3.4.1 MATERIALS AND EQUIPMENTS USED

(i) Mould (Internal Dia. : 25.7 cm , Height: 30.1 cm)

[The steel cylindrical tank was designed big enough to avoid boundary effect on bearing capacity.]

Figure 3.3 Mould used for specimen preparation

(ii) **Cover Plate (thickness: 1.7 cm, dia.: 25.6 cm)**

(iii) **Trays**

(iv) **Shaker**

(v) **Proving Ring (No. PR. 5 KN.0256, No. PR 1 KN. 0292)**

Figure 3.4 Proving Ring

(vi) **Dial Gauge (least count 0.01 mm)**

Figure 3.5 Dial Gauge

(vii) Loading Frame

Figure 3.6 Loading Frame at NIT ROURKELA

3.4.2 EXPERIMENTAL SETUP

- All the engineering properties of the sand were determined with the help of experiments as mentioned earlier.
- Weight of sand required to be filled in the given mould for different relative densities were calculated.
- Specimens were prepared for each relative density.
- The model footing was made out of steel plate of 5 mm thickness and diameter 45 mm. It had a smooth bottom face and a hole at the center of the top face for mounting the proving ring.

Figure 3.7 Compaction on the Shaker

Figure 3.8 Compacted Specimen

- A dial gauges was attached along the centre-line on both sides of the footing to measure the displacement (settlement).
- The sand was placed in the mould and compacted to attain desired relative density.
- Fiber to be added in the sand was considered as a part of the solids fraction in the void-solid matrix of the soil.
- The designated fibers were weighed according to the pre-determined reinforcement content and mixed into the sand in small parts until all of the fibers were effectively distribution within the sand. The fibers were mixed thoroughly by hand to get a fairly uniform mixture. During mixing, a segregation or floating tendency of fibers was noted. Much care was taken to mix the fibers uniformly. With increase in fiber content, difficulty in mixing also increased. However, it was possible to get a acceptably uniform fiber-sand mixture.

- The specimen prepared is placed on the loading frame and load is applied by strain-controlled gear mechanism. The rate of strain applied was 1.2 mm/min. The resistance offered by the footing(Bearing pressure of the footing) was noted from the dial gauge of the proving ring at different penetration values. Hence Stress strain curve for different relative densities with and without reinforcement was determined.

3.4.3 TEST VARIABLES

3.4.3.1 Fiber Content

Specimens for different fiber contents were prepared. In this study fiber contents of 0%, 0.1%, 0.25%, 0.5%, 0.75% were considered.

3.4.3.2 Relative Density

Specimens for different relative densities were prepared. In this project work relative densities of 40%, 55%, 73%, 88% were considered.

Four specimens for each case were prepared and best three results were averaged and plotted.

Chapter 4

RESULTS

&

DISCUSSIONS

4.1 RESULTS & DISCUSSIONS

The stress-strain curves obtained from the tests for unreinforced sand & reinforced sand with the fiber content of 0.1% to 0.75% and with different relative densities are shown in Figures 4.1 – 4.9. The test results obtained suggest that the fiber reinforcements can change the brittle behavior of the sand to a comparatively ductile one. That means, the samples tested with fiber inclusions exhibited a smaller loss of post-peak strength. This reduction in the loss of post-peak strength is magnified for higher vertical stresses and fiber contents.

Curves of Bearing Capacity- Fiber Content, Bearing Capacity- Relative Density were also plotted to display the variation of bearing capacity with inclusion of reinforcements.

Curves of Bearing Capacity Ratio- Fiber Content, Bearing Capacity Ratio - Relative Density were also plotted. These show the magnification in bearing capacity with introduction of fiber reinforcement.

TABLE 4.1 (Data Sheet for the specimens having fiber content = 0%)

		R.D. = 88%	R.D. = 73%	R.D. = 55%	R.D. = 40%
Settlement (in mm)	**% strain**	**Stress (KPa)**	**Stress (KPa)**	**Stress (KPa)**	**Stress (KPa)**
0	0	0	0	0	0
0.5	1.1	158.36	19.79	25.17	4.40
1	2.2	239.37	47.41	59.14	25.66
1.5	3.3	276.76	75.02	76.61	43.99
2	4.4	280.42	91.89	87.98	69.65
2.5	5.6	250.37	93.84	92.62	84.31
3	6.7	195.01	93.11	90.42	87.24
3.5	7.8	153.96	87.73	85.29	81.38
4	8.9	126.47	83.09	78.69	73.31
4.5	10.0	108.14	79.91	75.88	71.11
5	11.1	92.01	76.25	74.17	61.58
5.5	12.2	89.44	72.82	70.87	57.92
6	13.3	82.11	71.11	68.79	50.59
6.5	14.4	74.05	70.38	67.20	46.92
7	15.6	77.35	69.40	67.20	42.52
7.5	16.7	76.25	69.40	64.88	40.32
8	17.8	76.98	68.91	64.03	35.19
8.5	18.9	75.51	69.65	65.13	38.12
9	20.0	76.61	71.85	66.23	40.32
9.5	21.1	78.45	73.07	66.47	41.06
10	22.2	86.88	75.02	66.84	46.92
10.5	23.3	92.74	76.49	69.28	48.39
11	24.4	100.44	78.69	70.75	47.65
11.5	25.6	103.01	79.18	71.72	49.12
12	26.7	109.24	81.13	73.68	50.59
12.5	27.8	118.77	83.09	74.17	54.25
13	28.9	124.27	85.04	75.64	54.25
13.5	30.0	127.57	85.29	77.35	57.92
14	31.1	133.06	86.27	78.20	58.65
14.5	32.2	134.90	88.46	80.40	61.58
15	33.3	139.30	92.37	81.13	64.52
15.5	34.4	146.99	95.06	80.89	65.98
16	35.6	152.49	97.26	82.48	68.18
16.5	36.7	160.92	100.93	83.46	71.11
17	37.8	164.59	102.64	83.58	75.51
17.5	38.9	165.69	106.79	87.73	76.25
18	40.0	172.65	110.00	90.05	78.45
18.5	41.1	175.95	113.00	91.64	81.38
19	42.2	188.05	121.00	93.11	80.64
19.5	43.3	192.45	129.00	94.45	84.31

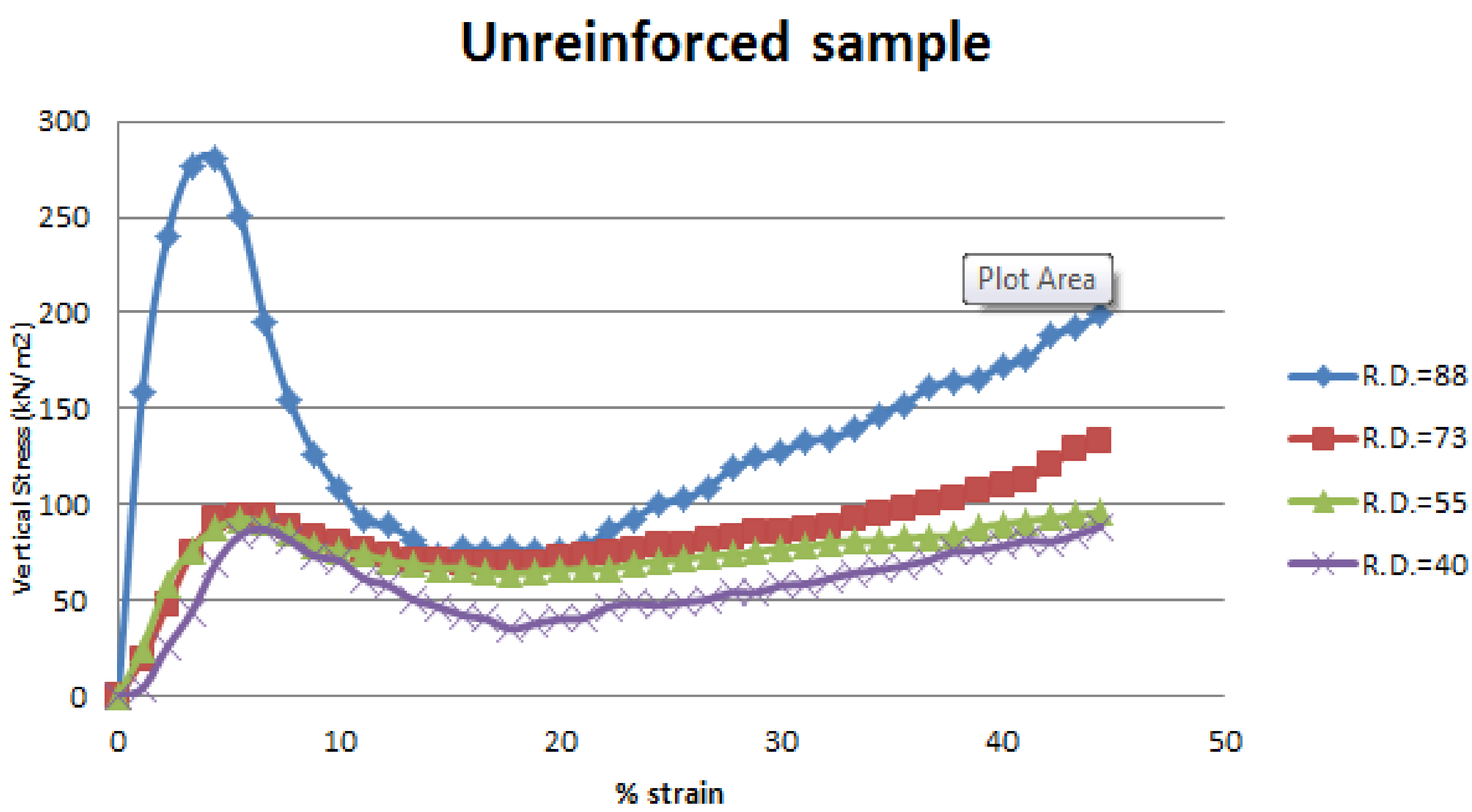

Figure 4.1: Stress-Strain Curve (fiber % = 0)

TABLE 4.2 (Data Sheet for the specimens having fiber content = 0.1%)

		R.D. = 88%	R.D. = 73%	R.D. = 55%	R.D. = 40%
Settlement (in mm)	% strain	Stress (KPa)	Stress (KPa)	Stress (KPa)	Stress (KPa)
0	0	0	0	0	0
0.5	1.1	109.97	43.25	27.13	14.66
1	2.2	186.95	90.91	44.72	25.66
1.5	3.3	250.00	127.57	58.65	32.26
2	4.4	301.68	151.76	66.72	35.92
2.5	5.6	314.15	161.29	70.38	37.39
3	6.7	297.65	157.62	65.25	36.66
3.5	7.8	251.83	153.96	65.25	35.19
4	8.9	213.71	144.43	63.05	35.19
4.5	10.0	162.76	143.69	62.32	33.72
5	11.1	139.66	140.76	61.58	32.26
5.5	12.2	131.60	134.90	61.58	29.33
6	13.3	122.80	126.83	60.85	28.59
6.5	14.4	115.84	121.70	60.85	28.59
7	15.6	111.80	120.97	62.32	31.52
7.5	16.7	104.11	122.43	63.05	33.72
8	17.8	107.04	115.10	62.32	33.72
8.5	18.9	107.04	112.17	62.32	36.66
9	20.0	104.11	117.30	65.25	37.39
9.5	21.1	104.11	117.30	63.78	38.86
10	22.2	109.60	123.17	65.25	38.86
10.5	23.3	113.64	126.10	65.98	40.32
11	24.4	121.33	122.43	65.98	39.59
11.5	25.6	126.83	126.10	66.72	40.32
12	26.7	129.76	123.90	65.98	40.32
12.5	27.8	131.96	122.43	65.25	41.06
13	28.9	134.16	129.03	69.65	41.79
13.5	30.0	141.49	133.43	70.38	43.25
14	31.1	145.16	137.83	74.78	45.45
14.5	32.2	151.39	145.16	74.05	48.39
15	33.3	159.82	148.09	76.98	52.05
15.5	34.4	165.32	151.03	79.18	52.79
16	35.6	173.02	151.76	79.91	53.52
16.5	36.7	179.62	156.16	81.38	52.79
17	37.8	185.48	158.36	82.11	52.05
17.5	38.9	192.45	164.96	84.31	54.25
18	40.0	197.58	164.22	88.71	54.25
18.5	41.1	200.51	170.09	89.44	55.72
19	42.2	209.31	173.75	92.37	56.45
19.5	43.3	213.34	176.69	98.24	58.65

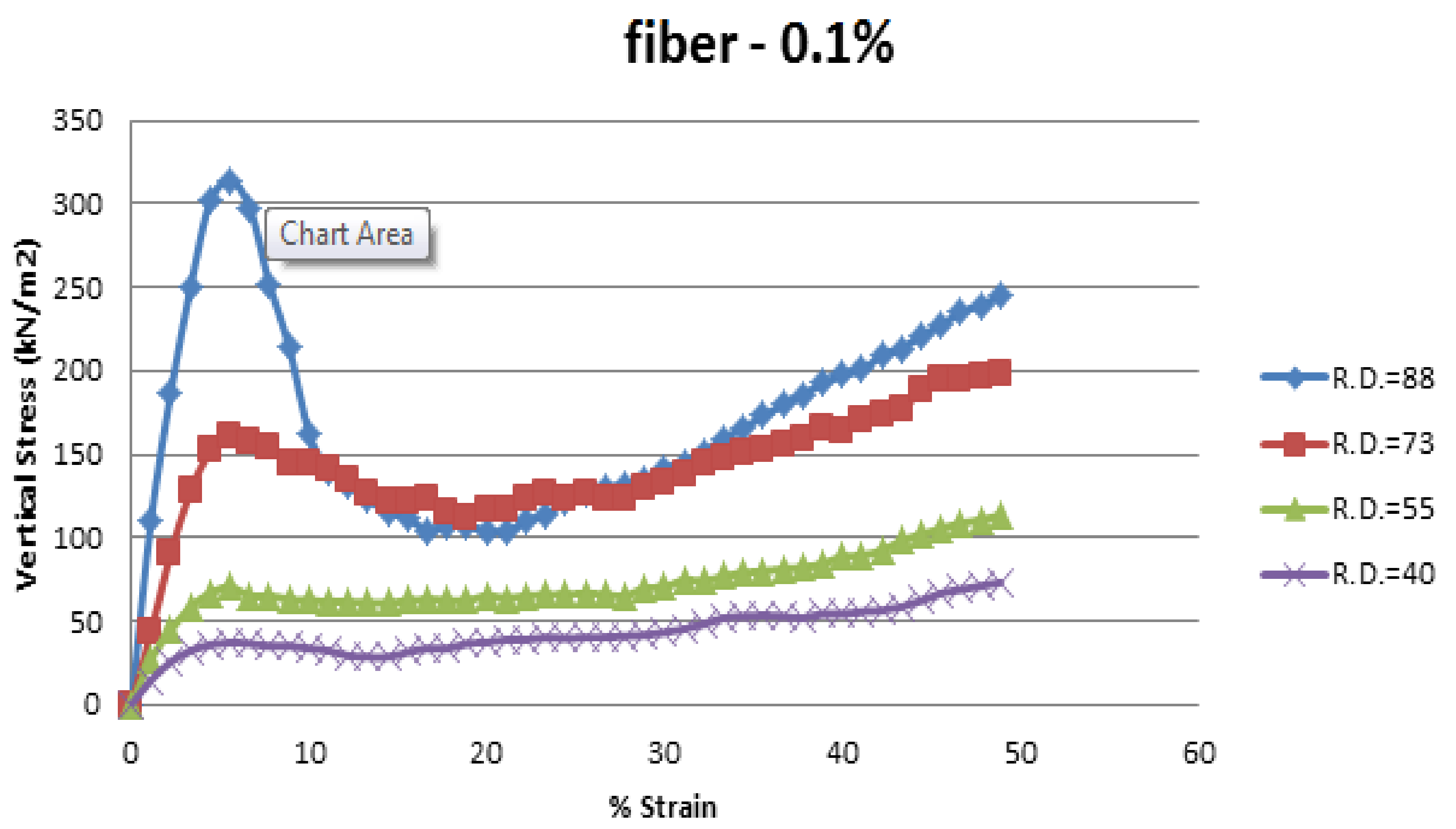

Figure 4.2: Stress-Strain Curve (Fiber % = 0.1%)

TABLE 4.3 (Data Sheet for the specimens having fiber content = 0.25%)

		R.D. = 88%	R.D. = 73%	R.D. = 55%	R.D. = 40%
Settlement (in mm)	% strain	Stress (KPa)	Stress (KPa)	Stress (KPa)	Stress (KPa)
0	0	0	0	0	0
0.5	1.1	0	0	0	0
1	2.2	39.22	46.92	36.66	22.73
1.5	3.3	87.98	92.37	70.38	37.76
2	4.4	131.96	140.40	90.91	46.92
2.5	5.6	166.05	185.48	107.40	52.79
3	6.7	193.18	222.51	120.23	56.08
3.5	7.8	208.94	250.37	126.83	57.18
4	8.9	211.14	263.56	125.37	54.62
4.5	10.0	203.44	279.32	124.27	52.42
5	11.1	196.11	290.69	120.97	51.32
5.5	12.2	181.08	288.49	116.57	51.32
6	13.3	176.32	274.93	112.90	50.59
6.5	14.4	160.19	267.23	112.17	50.59
7	15.6	151.76	265.03	109.24	48.39
7.5	16.7	151.03	259.90	108.50	49.49
8	17.8	149.19	254.76	107.04	51.69
8.5	18.9	149.93	245.23	108.14	53.89
9	20.0	151.39	235.34	109.97	54.99
9.5	21.1	152.49	234.60	110.70	54.62
10	22.2	155.42	234.60	112.17	56.45
10.5	23.3	159.09	240.10	112.54	57.18
11	24.4	158.36	243.40	112.54	58.28
11.5	25.6	163.12	245.23	114.00	59.75
12	26.7	167.89	245.97	114.74	60.12
12.5	27.8	169.72	248.17	117.67	61.95
13	28.9	175.95	254.76	115.10	62.68
13.5	30.0	176.32	259.53	117.30	64.88
14	31.1	181.08	262.83	120.97	66.72
14.5	32.2	185.48	266.49	123.90	70.38
15	33.3	189.88	267.59	129.03	73.68
15.5	34.4	198.31	272.36	131.96	77.35
16	35.6	201.98	269.43	137.83	78.45
16.5	36.7	207.11	282.99	138.93	79.91
17	37.8	212.24	293.99	142.59	78.45
17.5	38.9	222.87	296.19	145.16	79.18
18	40.0	222.14	311.22	146.63	80.64
18.5	41.1	229.10	320.75	147.36	81.38
19	42.2	233.50	336.14	149.93	85.41
19.5	43.3	242.30	353.00	150.66	87.98

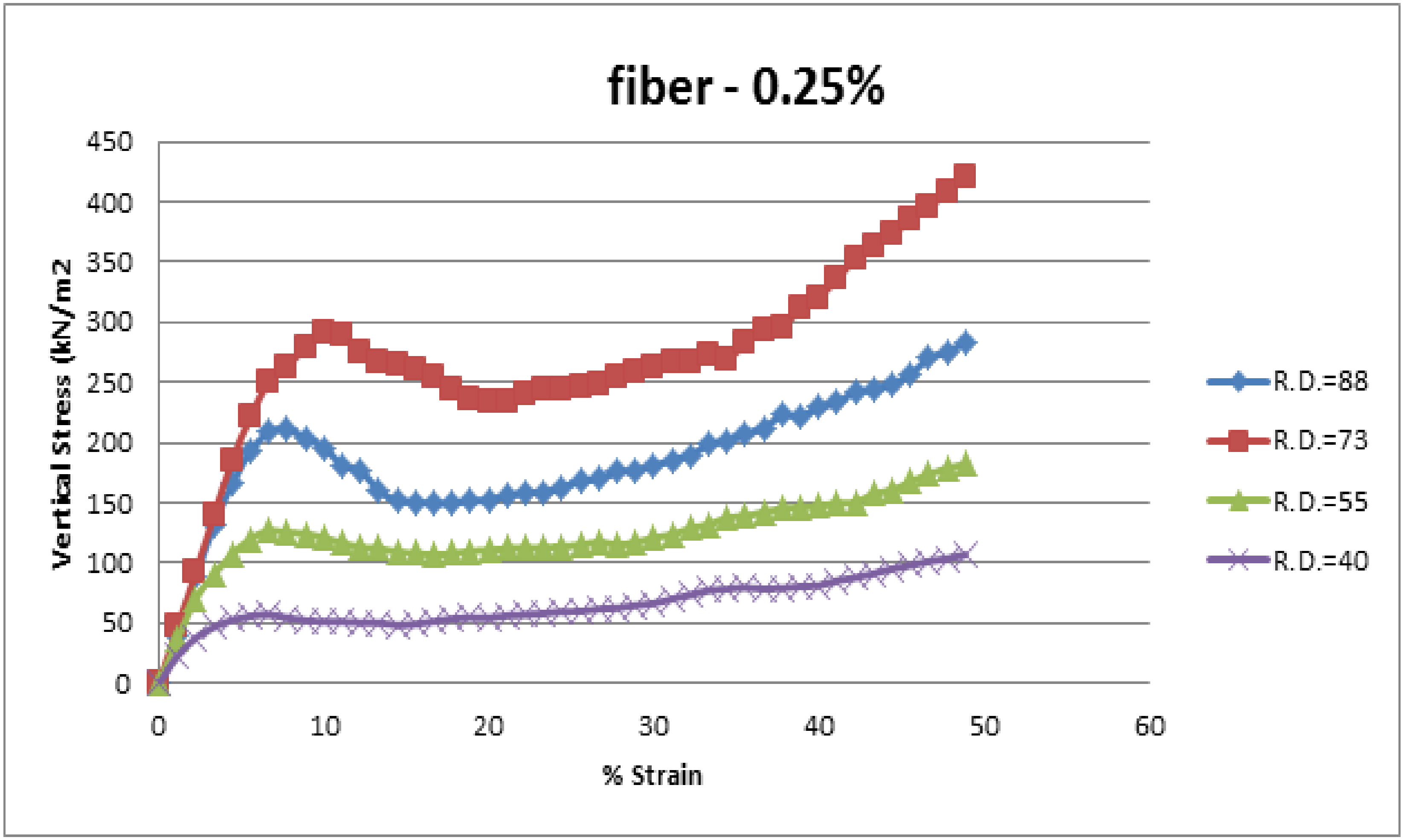

Figure 4.3: Stress-Strain Curve (Fiber % = 0.25%)

TABLE 4.4 (Data Sheet for the specimens having fiber content = 0.5%)

		R.D. = 88%	R.D. = 73%	R.D. = 55%	R.D. = 40%
Settlement (in mm)	% strain	Stress (KPa)	Stress (KPa)	Stress (KPa)	Stress (KPa)
0	0	0	0	0	0
0.5	1.1	45.66	63.64	5.87	28.59
1	2.2	113.37	121.47	46.19	51.32
1.5	3.3	217.30	173.85	80.64	74.41
2	4.4	319.65	222.21	112.90	95.31
2.5	5.6	436.17	267.93	147.36	114.37
3	6.7	544.82	301.38	178.15	133.80
3.5	7.8	637.72	324.11	201.61	150.29
4	8.9	719.60	341.73	222.87	166.42
4.5	10.0	773.14	356.56	236.80	182.55
5	11.1	793.61	366.74	239.73	194.28
5.5	12.2	785.74	370.95	247.07	204.18
6	13.3	757.39	372.83	246.33	217.37
6.5	14.4	730.63	372.15	249.27	226.91
7	15.6	695.98	369.84	248.53	238.64
7.5	16.7	664.49	365.36	244.87	244.13
8	17.8	640.87	355.15	245.60	251.47
8.5	18.9	620.40	347.60	248.53	255.13
9	20.0	599.93	346.41	251.47	259.16
9.5	21.1	579.46	340.30	253.66	261.73
10	22.2	565.29	332.48	252.93	264.29
10.5	23.3	557.42	323.58	253.66	268.33
11	24.4	540.10	320.02	248.53	274.93
11.5	25.6	533.80	317.44	250.73	280.06
12	26.7	516.48	316.81	255.86	280.06
12.5	27.8	507.03	315.46	255.13	281.52
13	28.9	508.60	313.26	260.26	286.66
13.5	30.0	503.88	311.16	269.06	295.09
14	31.1	514.90	316.41	272.73	300.22
14.5	32.2	524.35	320.78	280.06	298.75
15	33.3	529.07	323.93	286.66	299.85
15.5	34.4	535.37	321.70	291.79	306.82
16	35.6	543.25	328.27	303.52	313.78
16.5	36.7	552.69	333.70	305.72	318.91
17	37.8	568.44	346.84	314.51	320.75
17.5	38.9	579.46	359.25	321.85	324.78
18	40.0	585.76	367.39	318.91	326.61
18.5	41.1	598.36	379.94	324.05	324.78
19	42.2	609.38	391.61	329.91	328.44
19.5	43.3	623.55	402.93	335.04	327.34

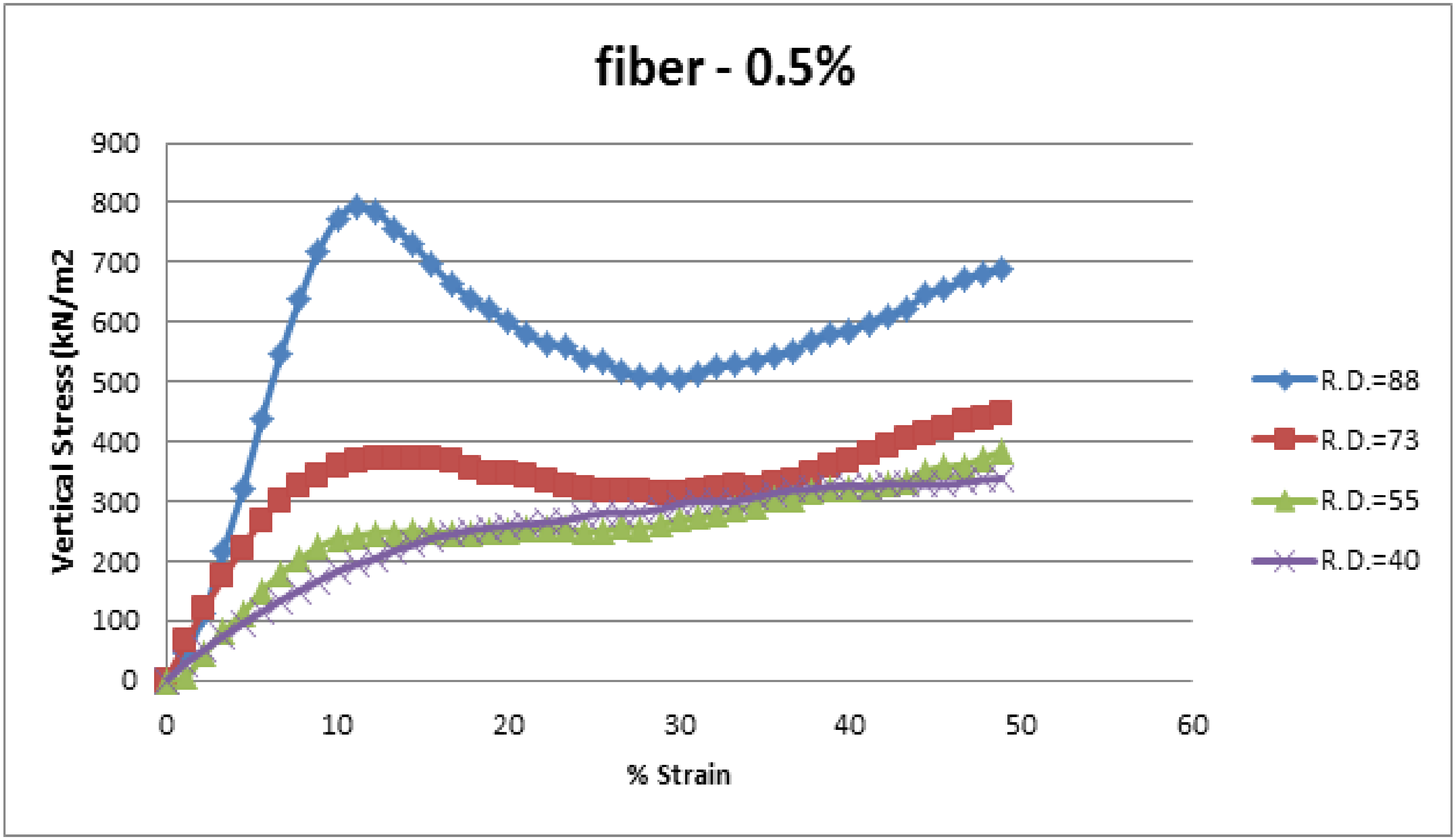

Figure 4.4: Stress-Strain Curve (Fiber % = 0.5%)

<u>TABLE 4.5 (</u>Data Sheet for the specimens having fiber content = 0.75%)

		R.D. = 88%	R.D. = 73%	R.D. = 55%	R.D. = 40%
Settlement (in mm)	% strain	Stress (KPa)	Stress (KPa)	Stress (KPa)	Stress (KPa)
0	0	0	0	0	0
0.5	1.1	66.13	66.13	71.85	30.79
1	2.2	122.82	136.99	115.10	60.12
1.5	3.3	193.68	203.13	160.56	88.71
2	4.4	278.71	264.54	186.95	117.30
2.5	5.6	377.91	321.22	206.74	147.36
3	6.7	472.39	340.12	225.81	175.95
3.5	7.8	566.87	344.84	233.87	201.61
4	8.9	666.07	335.40	236.07	229.47
4.5	10.0	751.10	321.22	241.93	253.66
5	11.1	864.47	302.33	239.73	268.33
5.5	12.2	973.12	283.43	236.07	284.46
6	13.3	1086.49	264.54	232.40	301.32
6.5	14.4	1195.14	240.92	222.87	304.25
7	15.6	1317.96	240.92	219.21	326.24
7.5	16.7	1417.16	245.64	224.34	336.51
8	17.8	1525.81	245.64	228.00	345.31
8.5	18.9	1634.46	250.37	226.54	365.10
9	20.0	1733.66	255.09	238.27	378.30
9.5	21.1	1823.42	264.54	252.20	380.50
10	22.2	1908.45	273.98	261.73	386.36
10.5	23.3	1988.75	288.16	272.73	392.96
11	24.4	2073.78	307.05	280.79	394.43
11.5	25.6	2154.09	325.95	301.32	401.76
12	26.7	2248.56	335.40	316.71	407.62
12.5	27.8	2343.04	354.29	339.44	413.49
13	28.9	2442.24	377.91	362.17	412.02
13.5	30.0	2536.72	406.25	378.30	421.55
14	31.1	2612.30	425.15	398.09	436.95
14.5	32.2	2678.44	444.04	420.82	442.08
15	33.3	2716.23	472.39	439.15	450.88
15.5	34.4	2768.19	486.56	451.61	458.94
16	35.6	2782.36	500.73	467.01	464.07
16.5	36.7	2787.09	524.35	473.60	469.21
17	37.8	2791.81	547.97	482.40	481.67
17.5	38.9	2805.98	566.87	494.87	498.53
18	40.0	2843.77	595.21	509.53	498.53
18.5	41.1	2839.05	628.28	513.19	509.53
19	42.2	2853.22	637.72	523.46	525.66
19.5	43.3	2857.94	670.79	545.45	538.85

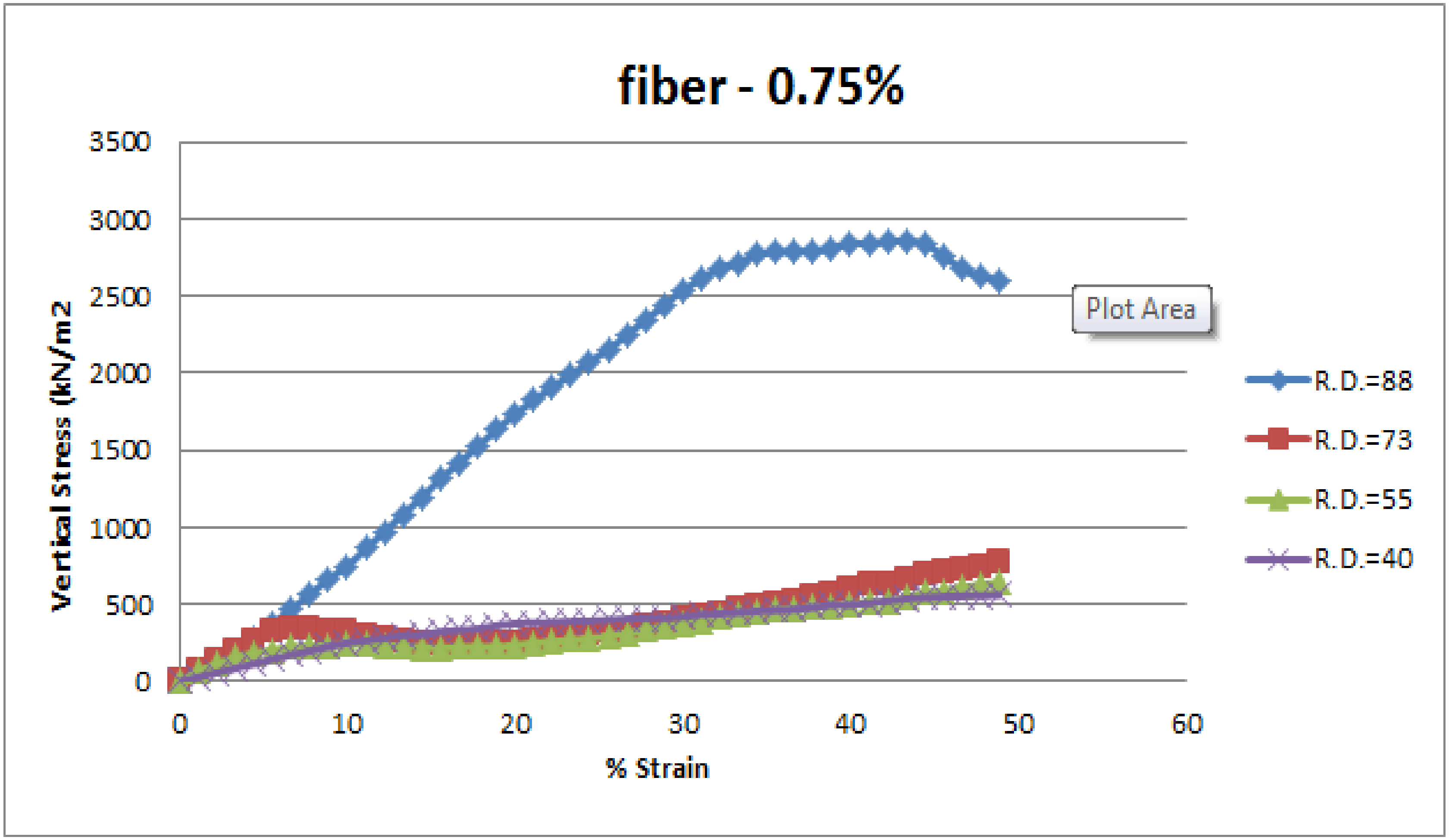

Figure 4.5 Stress-Strain Curve (Fiber % = 0.75%)

TABLE 4.6 (Data Sheet for the specimens having Relative Density = 88%)

	Fiber = 0%	Fiber = 0.1%	Fiber = 0.25%	Fiber = 0.5%	Fiber = 0.75%
Strain %	Stress (KPa)	Stress (KPa)	Stress (KPa)	Stress (KPa)	Stress (KPa)
0	0	0	0	0	0
1.11	158.36	109.97	39.22	45.66	66.13
2.22	239.37	186.95	87.98	113.37	122.82
3.33	276.76	250.00	131.96	217.30	193.68
4.44	280.42	301.68	166.05	319.65	278.71
5.56	250.37	314.15	193.18	436.17	377.91
6.67	195.01	297.65	208.94	544.82	472.39
7.78	153.96	251.83	211.14	637.72	566.87
8.89	126.47	213.71	203.44	719.60	666.07
10.00	108.14	162.76	196.11	773.14	751.10
11.11	92.01	139.66	181.08	793.61	864.47
12.22	89.44	131.60	176.32	785.74	973.12
13.33	82.11	122.80	160.19	757.39	1086.49
14.44	74.05	115.84	151.76	730.63	1195.14
15.56	77.35	111.80	151.03	695.98	1317.96
16.67	76.25	104.11	149.19	664.49	1417.16
17.78	76.98	107.04	149.93	640.87	1525.81
18.89	75.51	107.04	151.39	620.40	1634.46
20.00	76.61	104.11	152.49	599.93	1733.66
21.11	78.45	104.11	155.42	579.46	1823.42
22.22	86.88	109.60	159.09	565.29	1908.45
23.33	92.74	113.64	158.36	557.42	1988.75
24.44	100.44	121.33	163.12	540.10	2073.78
25.56	103.01	126.83	167.89	533.80	2154.09
26.67	109.24	129.76	169.72	516.48	2248.56
27.78	118.77	131.96	175.95	507.03	2343.04
28.89	124.27	134.16	176.32	508.60	2442.24
30.00	127.57	141.49	181.08	503.88	2536.72
31.11	133.06	145.16	185.48	514.90	2612.30
32.22	134.90	151.39	189.88	524.35	2678.44
33.33	139.30	159.82	198.31	529.07	2716.23
34.44	146.99	165.32	201.98	535.37	2768.19
35.56	152.49	173.02	207.11	543.25	2782.36
36.67	160.92	179.62	212.24	552.69	2787.09
37.78	164.59	185.48	222.87	568.44	2791.81
38.89	165.69	192.45	222.14	579.46	2805.98
40.00	172.65	197.58	229.10	585.76	2843.77
41.11	175.95	200.51	233.50	598.36	2839.05
42.22	188.05	209.31	242.30	609.38	2853.22
43.33	192.45	213.34	243.77	623.55	2857.94

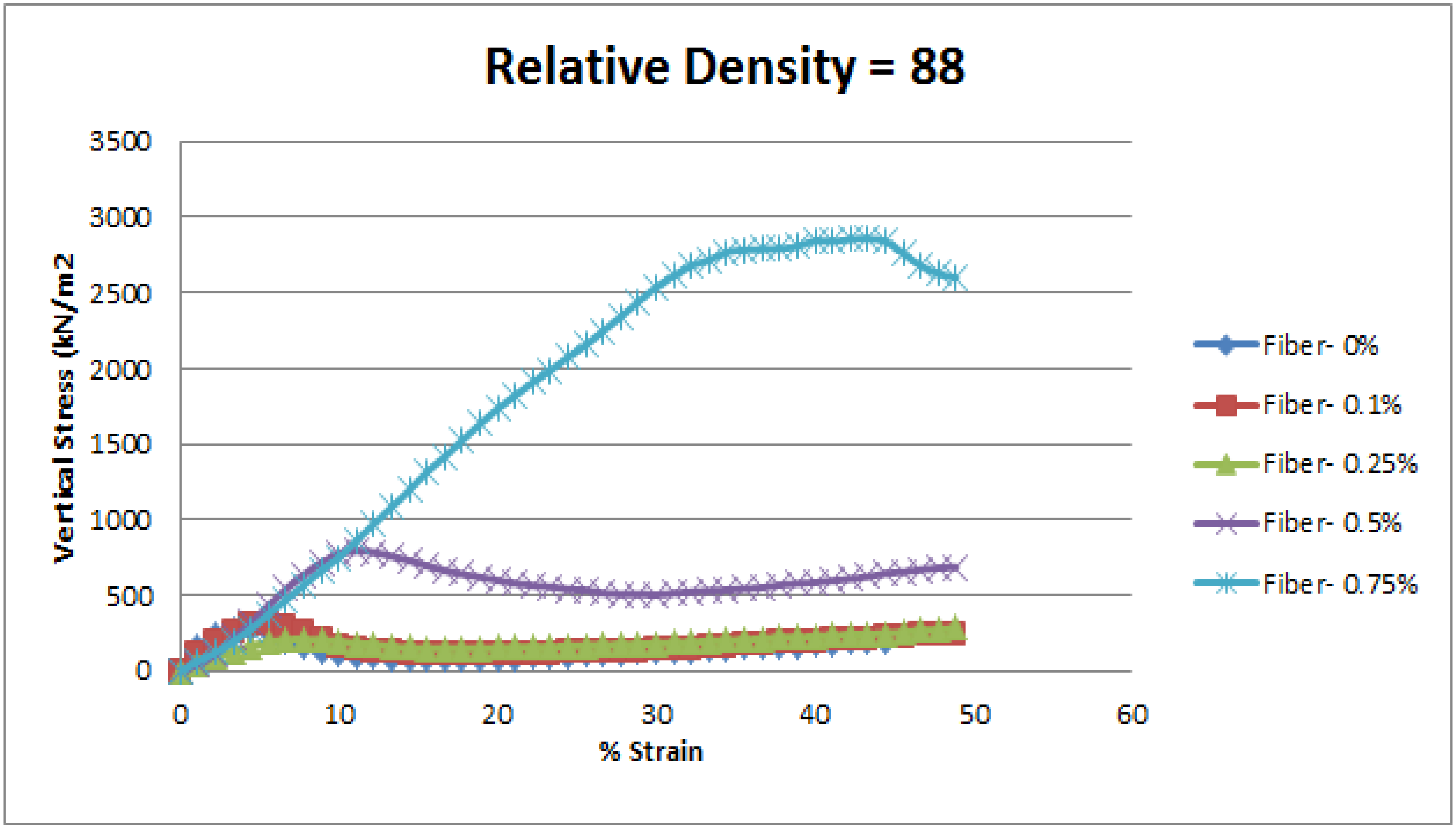

Figure 4.6 Stress- Strain Curve (Relative Density = 88%)

TABLE 4.7 (Data Sheet for the specimens having Relative Density = 73%)

	Fiber = 0%	Fiber = 0.1%	Fiber = 0.25%	Fiber = 0.5%	Fiber = 0.75%
Strain %	Stress (KPa)	Stress (KPa)	Stress (KPa)	Stress (KPa)	Stress (KPa)
0	0	0	0	0	0
1.11	19.79	43.25	46.92	63.64	66.13
2.22	47.41	90.91	92.37	121.47	136.99
3.33	75.02	127.57	140.40	173.85	203.13
4.44	91.89	151.76	185.48	222.21	264.54
5.56	93.84	161.29	222.51	267.93	321.22
6.67	93.11	157.62	250.37	301.38	340.12
7.78	87.73	153.96	263.56	324.11	344.84
8.89	83.09	144.43	279.32	341.73	335.40
10.00	79.91	143.69	290.69	356.56	321.22
11.11	76.25	140.76	288.49	366.74	302.33
12.22	72.82	134.90	274.93	370.95	283.43
13.33	71.11	126.83	267.23	372.83	264.54
14.44	70.38	121.70	265.03	372.15	240.92
15.56	69.40	120.97	259.90	369.84	240.92
16.67	69.40	122.43	254.76	365.36	245.64
17.78	68.91	115.10	245.23	355.15	245.64
18.89	69.65	112.17	235.34	347.60	250.37
20.00	71.85	117.30	234.60	346.41	255.09
21.11	73.07	117.30	234.60	340.30	264.54
22.22	75.02	123.17	240.10	332.48	273.98
23.33	76.49	126.10	243.40	323.58	288.16
24.44	78.69	122.43	245.23	320.02	307.05
25.56	79.18	126.10	245.97	317.44	325.95
26.67	81.13	123.90	248.17	316.81	335.40
27.78	83.09	122.43	254.76	315.46	354.29
28.89	85.04	129.03	259.53	313.26	377.91
30.00	85.29	133.43	262.83	311.16	406.25
31.11	86.27	137.83	266.49	316.41	425.15
32.22	88.46	145.16	267.59	320.78	444.04
33.33	92.37	148.09	272.36	323.93	472.39
34.44	95.06	151.03	269.43	321.70	486.56
35.56	97.26	151.76	282.99	328.27	500.73
36.67	100.93	156.16	293.99	333.70	524.35
37.78	102.64	158.36	296.19	346.84	547.97
38.89	106.79	164.96	311.22	359.25	566.87
40.00	110.00	164.22	320.75	367.39	595.21
41.11	113.00	170.09	336.14	379.94	628.28
42.22	121.00	173.75	353.00	391.61	637.72
43.33	129.00	176.69	363.27	402.93	670.79

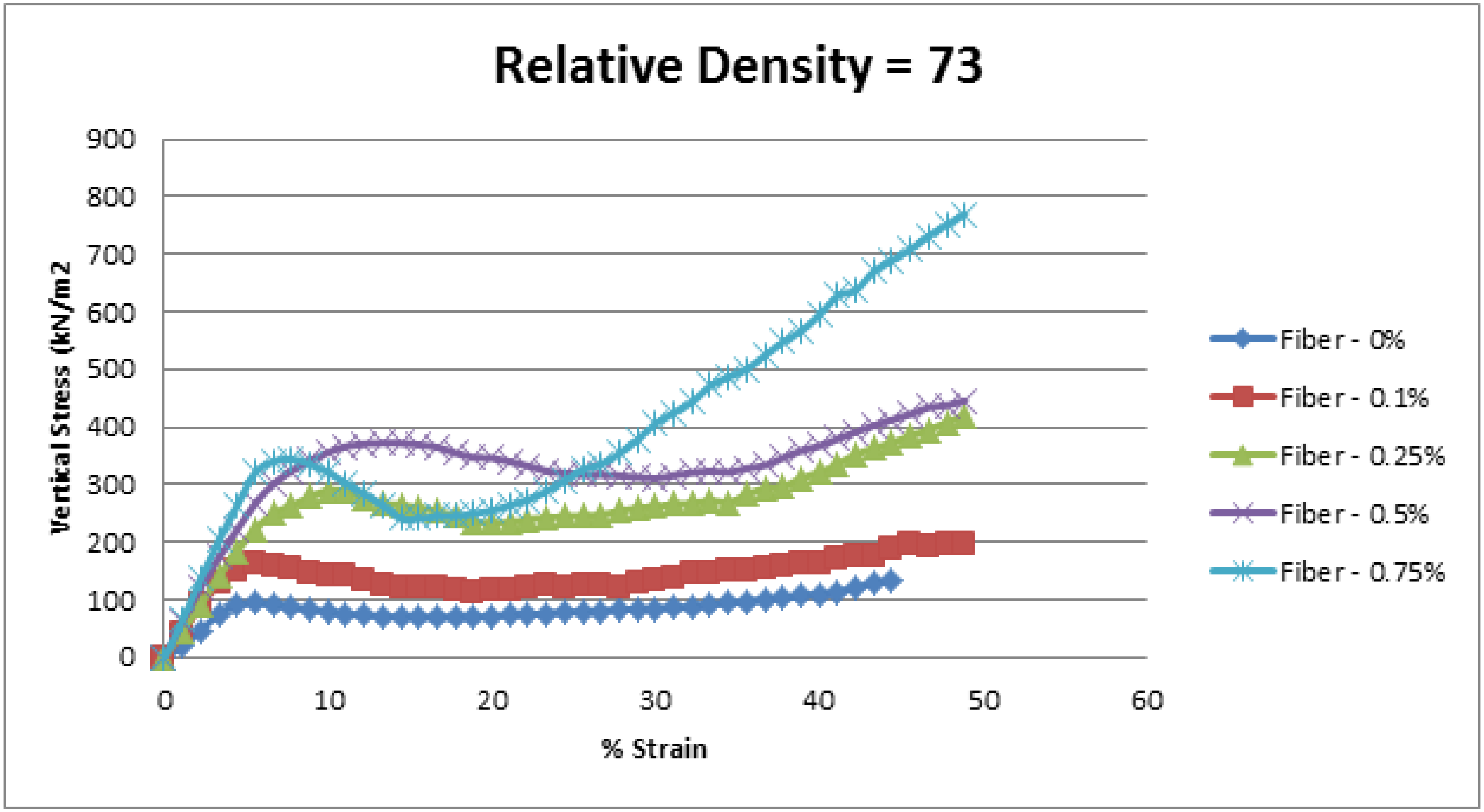

Figure 4.7 Stress- Strain Curve (Relative Density = 73%)

TABLE 4.8 (Data Sheet for the specimens having Relative Density = 55%)

	Fiber = 0%	Fiber = 0.1%	Fiber = 0.25%	Fiber = 0.5%	Fiber = 0.75%
Strain %	**Stress (KPa)**	**Stress (KPa)**	**Stress (KPa)**	**Stress (KPa)**	**Stress (KPa)**
0	0	0	0	0	0
1.11	25.17	27.13	36.66	5.87	71.85
2.22	59.14	44.72	70.38	46.19	115.10
3.33	76.61	58.65	90.91	80.64	160.56
4.44	87.98	66.72	107.40	112.90	186.95
5.56	92.62	70.38	120.23	147.36	206.74
6.67	90.42	65.25	126.83	178.15	225.81
7.78	85.29	65.25	125.37	201.61	233.87
8.89	78.69	63.05	124.27	222.87	236.07
10.00	75.88	62.32	120.97	236.80	241.93
11.11	74.17	61.58	116.57	239.73	239.73
12.22	70.87	61.58	112.90	247.07	236.07
13.33	68.79	60.85	112.17	246.33	232.40
14.44	67.20	60.85	109.24	249.27	222.87
15.56	67.20	62.32	108.50	248.53	219.21
16.67	64.88	63.05	107.04	244.87	224.34
17.78	64.03	62.32	108.14	245.60	228.00
18.89	65.13	62.32	109.97	248.53	226.54
20.00	66.23	65.25	110.70	251.47	238.27
21.11	66.47	63.78	112.17	253.66	252.20
22.22	66.84	65.25	112.54	252.93	261.73
23.33	69.28	65.98	112.54	253.66	272.73
24.44	70.75	65.98	114.00	248.53	280.79
25.56	71.72	66.72	114.74	250.73	301.32
26.67	73.68	65.98	117.67	255.86	316.71
27.78	74.17	65.25	115.10	255.13	339.44
28.89	75.64	69.65	117.30	260.26	362.17
30.00	77.35	70.38	120.97	269.06	378.30
31.11	78.20	74.78	123.90	272.73	398.09
32.22	80.40	74.05	129.03	280.06	420.82
33.33	81.13	76.98	131.96	286.66	439.15
34.44	80.89	79.18	137.83	291.79	451.61
35.56	82.48	79.91	138.93	303.52	467.01
36.67	83.46	81.38	142.59	305.72	473.60
37.78	83.58	82.11	145.16	314.51	482.40
38.89	87.73	84.31	146.63	321.85	494.87
40.00	90.05	88.71	147.36	318.91	509.53
41.11	91.64	89.44	149.93	324.05	513.19
42.22	93.11	92.37	150.66	329.91	523.46
43.33	94.45	98.24	157.62	335.04	545.45

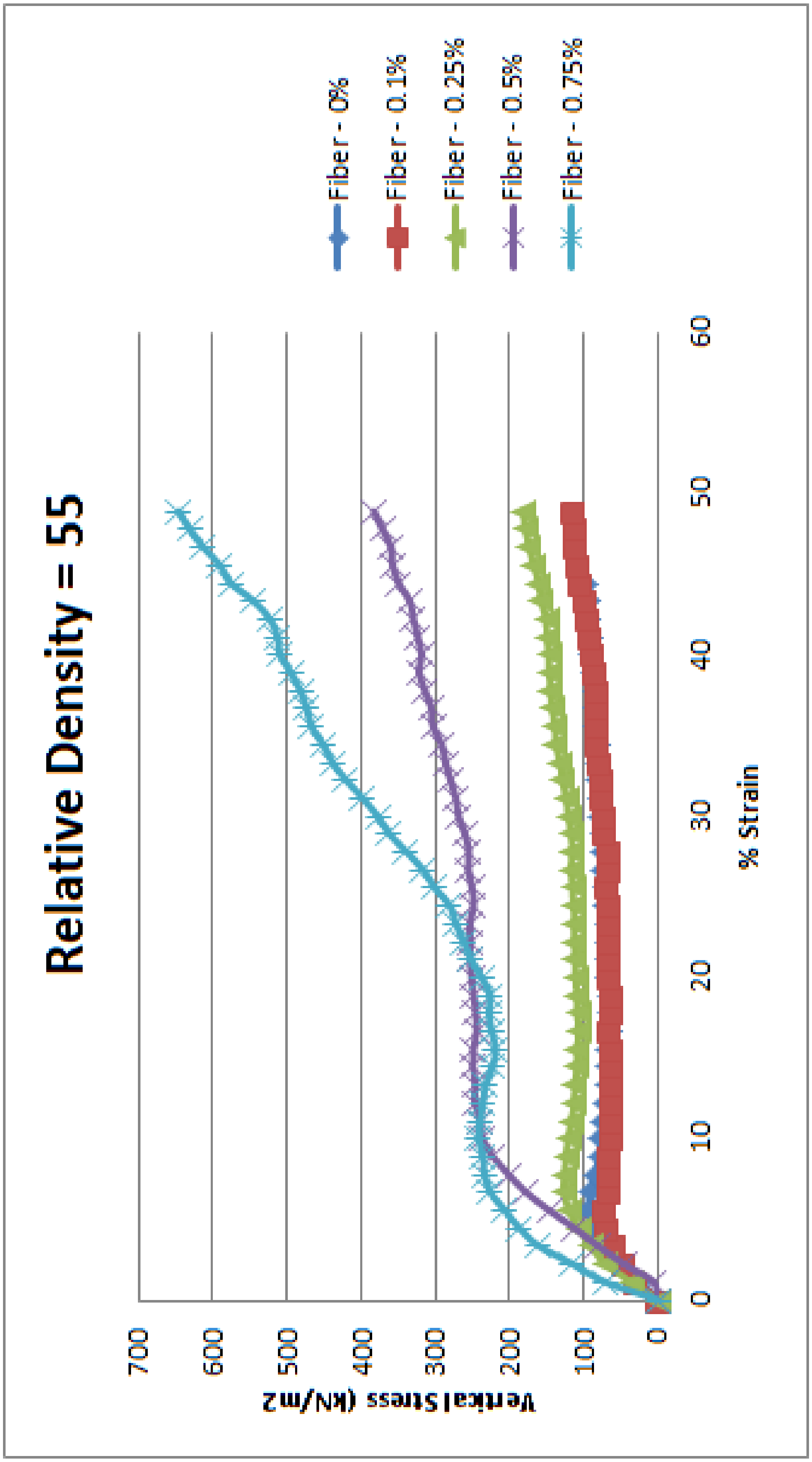

Figure 4.8 Stress- Strain Curve (Relative Density = 55%)

TABLE 4.9 (Data Sheet for the specimens having Relative Density = 40%)

	Fiber = 0%	Fiber = 0.1%	Fiber = 0.25%	Fiber = 0.5%	Fiber = 0.75%
Strain %	Stress (KPa)	Stress (KPa)	Stress (KPa)	Stress (KPa)	Stress (KPa)
0	0	0	0	0	0
1.11	4.40	14.66	22.73	28.59	30.79
2.22	25.66	25.66	37.76	51.32	60.12
3.33	43.99	32.26	46.92	74.41	88.71
4.44	69.65	35.92	52.79	95.31	117.30
5.56	84.31	37.39	56.08	114.37	147.36
6.67	87.24	36.66	57.18	133.80	175.95
7.78	81.38	35.19	54.62	150.29	201.61
8.89	73.31	35.19	52.42	166.42	229.47
10.00	71.11	33.72	51.32	182.55	253.66
11.11	61.58	32.26	51.32	194.28	268.33
12.22	57.92	29.33	50.59	204.18	284.46
13.33	50.59	28.59	50.59	217.37	301.32
14.44	46.92	28.59	48.39	226.91	304.25
15.56	42.52	31.52	49.49	238.64	326.24
16.67	40.32	33.72	51.69	244.13	336.51
17.78	35.19	33.72	53.89	251.47	345.31
18.89	38.12	36.66	54.99	255.13	365.10
20.00	40.32	37.39	54.62	259.16	378.30
21.11	41.06	38.86	56.45	261.73	380.50
22.22	46.92	38.86	57.18	264.29	386.36
23.33	48.39	40.32	58.28	268.33	392.96
24.44	47.65	39.59	59.75	274.93	394.43
25.56	49.12	40.32	60.12	280.06	401.76
26.67	50.59	40.32	61.95	280.06	407.62
27.78	54.25	41.06	62.68	281.52	413.49
28.89	54.25	41.79	64.88	286.66	412.02
30.00	57.92	43.25	66.72	295.09	421.55
31.11	58.65	45.45	70.38	300.22	436.95
32.22	61.58	48.39	73.68	298.75	442.08
33.33	64.52	52.05	77.35	299.85	450.88
34.44	65.98	52.79	78.45	306.82	458.94
35.56	68.18	53.52	79.91	313.78	464.07
36.67	71.11	52.79	78.45	318.91	469.21
37.78	75.51	52.05	79.18	320.75	481.67
38.89	76.25	54.25	80.64	324.78	498.53
40.00	78.45	54.25	81.38	326.61	498.53
41.11	81.38	55.72	85.41	324.78	509.53
42.22	80.64	56.45	87.98	328.44	525.66
43.33	84.31	58.65	91.28	327.34	538.85

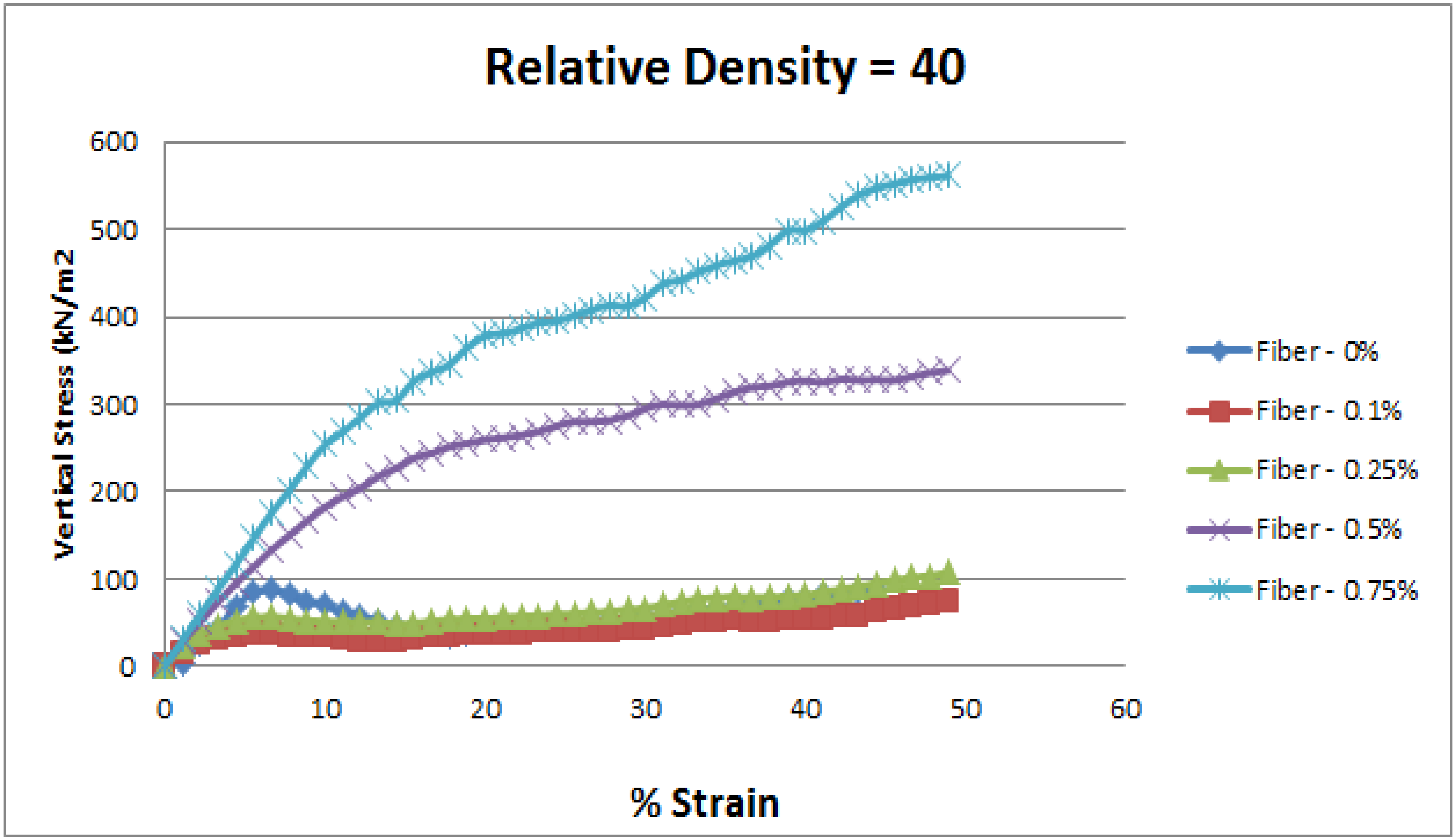

Figure 4.9 Stress- Strain Curve (Relative Density = 40%)

TABLE 4.10 (Variation of bearing capacity with fiber % for different Relative Densities)

	Bearing Capacity			
Fiber %	**R.D. = 40%**	**R.D. = 55%**	**R.D. = 73%**	**R.D. = 88%**
0	**87.25**	**92.61**	**95.8**	**280.4**
0.1	**40.15**	**70.38**	**161.28**	**314.15**
0.25	**57.8**	**126.83**	**208.95**	**290.71**
0.5	**259.16**	**247.07**	**370.95**	**793.61**
0.75	**378.3**	**239.27**	**344.84**	**1733.66**

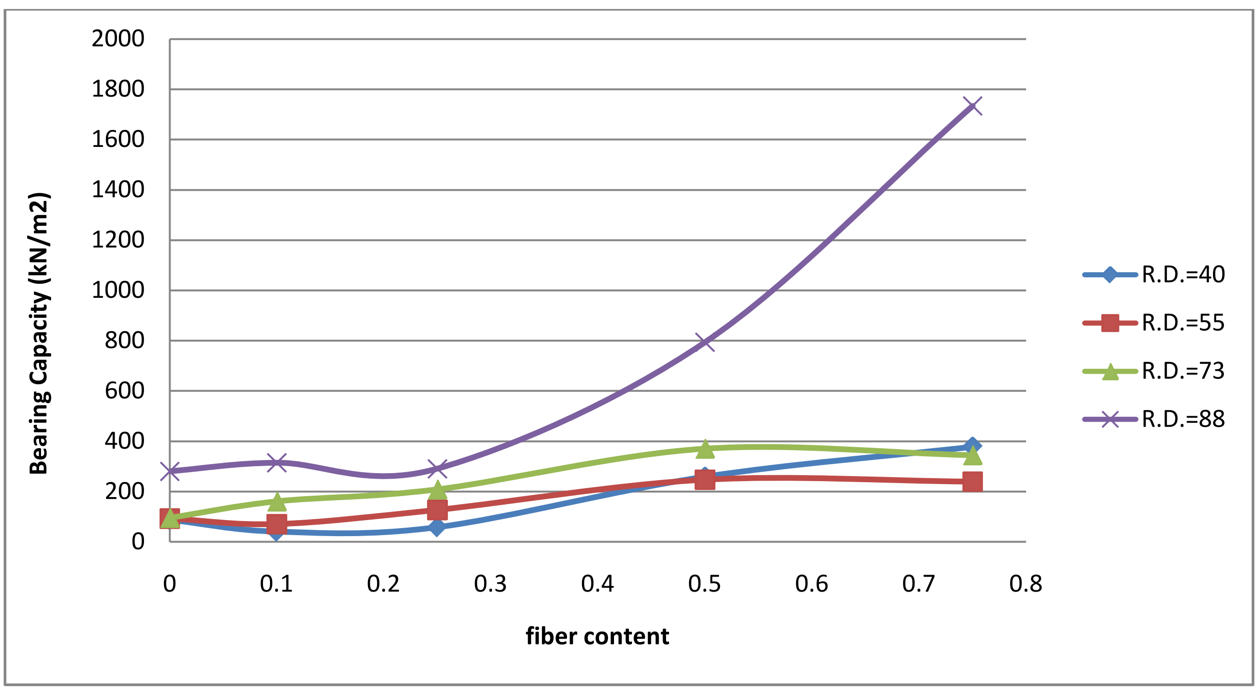

Figure 4.10 Bearing Capacities vs. Fiber Content

TABLE 4.11 (Variation of bearing capacity with Relative Densities for different fiber %)

	Bearing Capacity				
R.D.	**Fiber = 0%**	**Fiber = 0.1%**	**Fiber = 0.25%**	**Fiber = 0.5%**	**Fiber = 0.75%**
40	**87.25**	**40.15**	**57.8**	**259.16**	**378.3**
55	**92.61**	**70.38**	**126.83**	**247.07**	**239.27**
73	**95.8**	**161.28**	**208.95**	**370.95**	**344.84**
88	**280.4**	**314.15**	**290.71**	**793.61**	**1733.66**

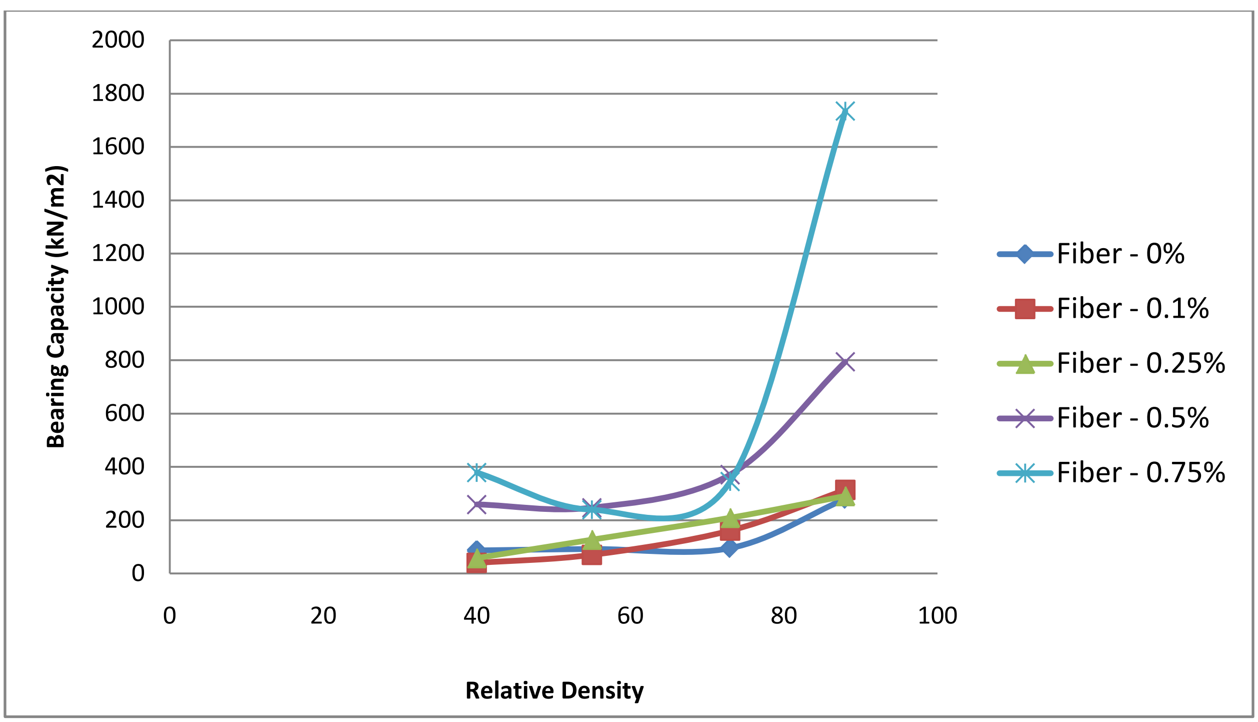

Figure 4.11 Bearing Capacities vs. Relative Density

TABLE 4.12 (Variation of bearing capacity ratio with fiber % for different Relative Densities)

	Bearing Capacity Ratio			
Fiber %	**R.D. = 40%**	**R.D. = 55%**	**R.D. = 73%**	**R.D. = 88%**
0	**1**	**1.061433**	**1.034446**	**2.926931**
0.1	**1**	**1.752927**	**2.29156**	**1.947855**
0.25	**1**	**2.194291**	**1.647481**	**1.39129**
0.5	**1**	**0.953349**	**1.501396**	**2.139399**
0.75	**1**	**0.632487**	**1.441217**	**5.027433**

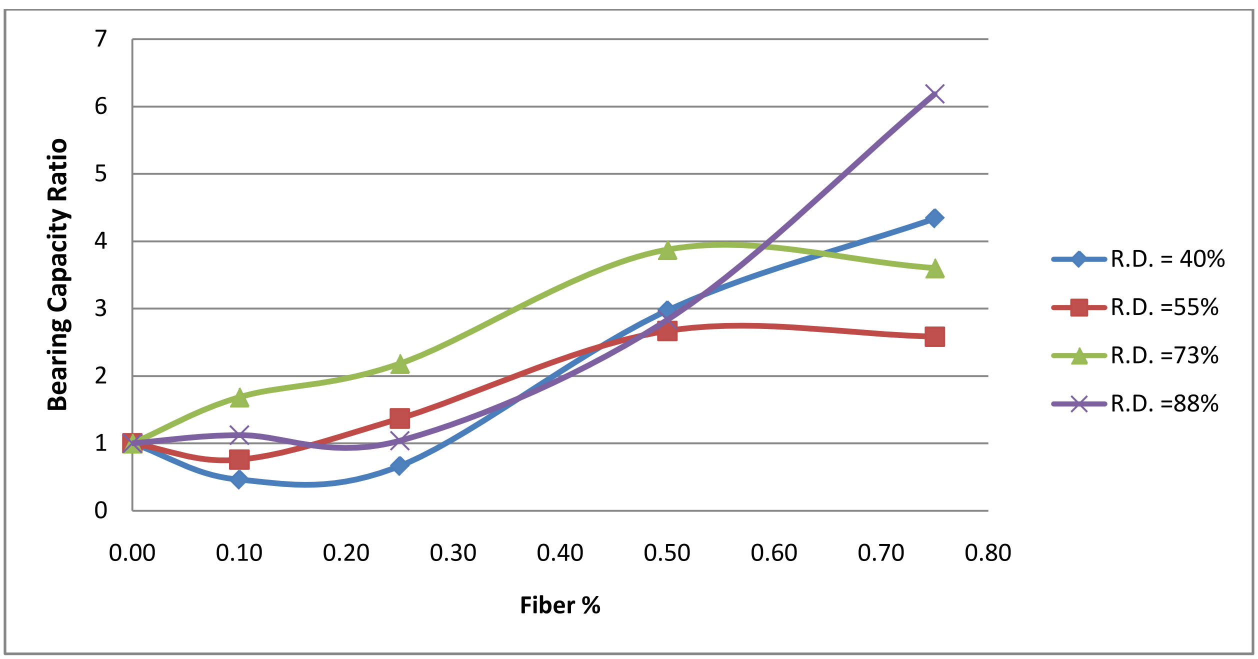

Figure 4.12 Bearing Capacities Ratio vs. Fiber Content

<u>TABLE 4.13 (</u>**Variation of bearing capacity ratio with Relative Densities for different fiber %)**

	Bearing Capacity Ratio				
R.D.	**Fiber = 0%**	**Fiber = 0.1%**	**Fiber = 0.25%**	**Fiber = 0.5%**	**Fiber = 0.75%**
40	**1**	**0.460172**	**0.662464**	**2.970315**	**4.335817**
55	**1**	**0.759961**	**1.369507**	**2.667854**	**2.58363**
73	**1**	**1.683507**	**2.181106**	**3.872129**	**3.599582**
88	**1**	**1.120364**	**1.036769**	**2.830278**	**6.18281**

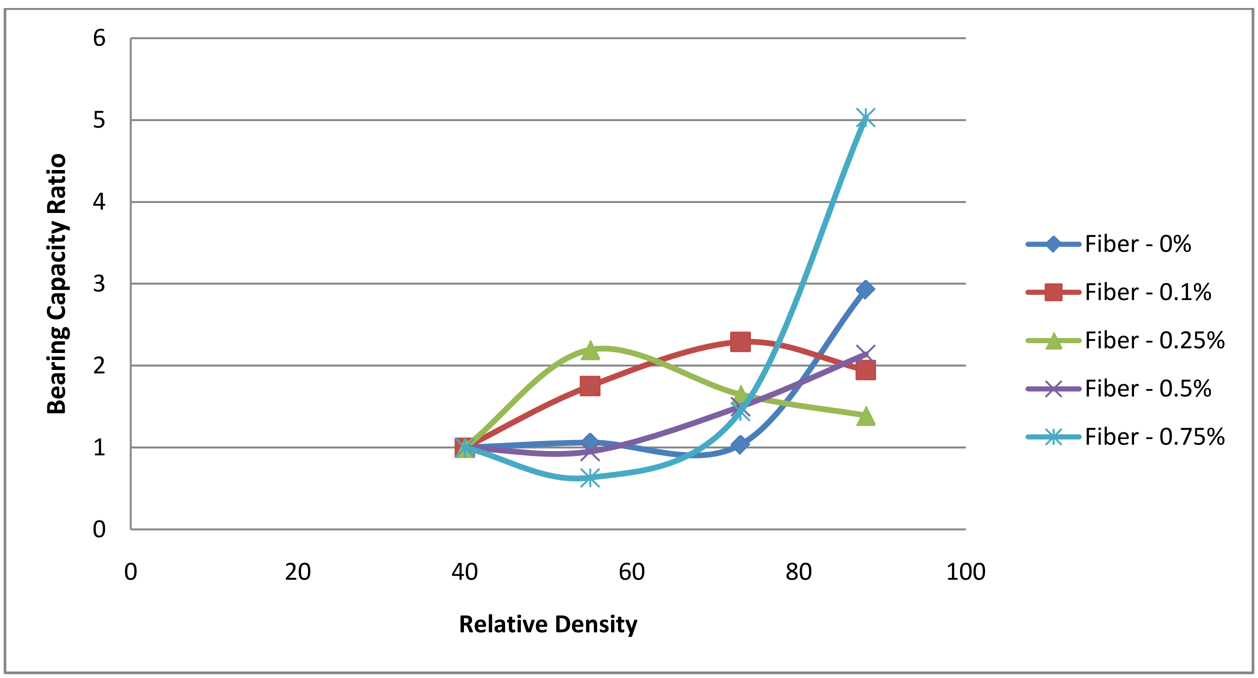

Figure 4.13 Bearing Capacities Ratio vs. Relative Density

Chapter 5

CONCLUSIONS

CONCLUSIONS

This study was undertaken to investigate the effect of fiber content on the bearing capacity of randomly distributed fiber-reinforced sand by measuring load-deformation.The following conclusions can be drawn from the experimental study.

- The bearing capacity of footings on randomly reinforced sand increases due to interference effects.
- Fiber reinforcements showed smaller loss of post-peak strength and changed the brittle behavior of the sand to a somewhat more ductile one. Hence, residual strength increases by adding the fiber reinforcements.
- Fiber reinforcements, having relatively low modulus, behave as “ideally extensible” inclusions.
- It is very effective in case of foundations subjected to dynamic and earthquake loading
- Reinforcement mobilization needs high strain, so in case of loose soil its effect is less as compared to dense soils

Chapter 6

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